PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 68 DECEMBER, 1942 No. 10

TECHNICAL PAPERS

AND

DISCUSSIONS

Published monthly, except July and August, at Prince and Lemon Streets, Lancaster, Pa., by the American Society of Civil Engineers. Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, September 23, 1937, at the Post Office at Lancaster, Pa., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy

CURRENT PAPERS AND DISCUSSIONS

		Discussion closes
Reduction of Mineral Content in Water with Organic Zeolites. *R. F. Goudey*	Feb., 1942	
Discussion	Feb., Apr., May, 1942	Closed*
Hydrodynamics of Model Storm Sewer Inlets Applied to Design. *G. S. Tapley*	Mar., 1942	
Discussion	Sept., Dec., 1942	Closed
Basic Economic Considerations Affecting Single and Multiple Purpose Projects. *Otis M. Page*	Mar., 1942	
Discussion	Sept., 1942	Closed*
Allocation of the Tennessee Valley Authority Projects. *Theodore B. Parker*	Dec., 1941	
Discussion	Feb., Mar., May, June, Dec., 1942	Closed*
Effect of Variation of Elastic Characteristics on Static Unknowns. *A. Hrennikoff*	Jan., 1942	
Discussion	Apr., May, Oct., 1942	Closed*
Unusual Events and Their Relation to Federal Water Policies. *W. G. Hoyt*	Feb., 1942	
Discussion	Apr., Sept., 1942	Closed*
Design of St. Georges Tied Arch Span. *J. M. Garrelts*	Dec., 1941	
Discussion	Feb., Apr., May, June, Sept., Oct., Nov., 1942	Closed*
The Grease Problem in Sewage Treatment. *Almon L. Fales and Samuel A. Greeley*	Feb., 1942	
Discussion	Feb., May, June, Sept., Nov., 1942	Closed*
Profile Curves for Open-Channel Flow. *Dwight F. Gunder*	Apr., 1942	
Discussion	June, Sept., Oct., Nov., 1942	Closed*
Model Tests on Structures for Hydroelectric Developments. *L. M. Davis*	Apr., 1942	
Discussion	Apr., June, 1942	Closed*
Resistance of Soft Fill to Static Wheel Loads. *W. Watters Pagon*	Feb., 1942	Jan. 1, 1943
Viscosity and Surface Tension Effects on V-Notch Weir Coefficients. *Arno T. Lenz*	Mar., 1942	
Discussion	June, Sept., 1942	Jan. 1, 1943
Early Contributions to Mississippi River Hydrology. *C. S. Jarvis*	Mar., 1942	
Discussion	Sept., Nov., Dec., 1942	Jan. 1, 1943
Stress Concentrations in Plates Loaded Over Small Areas. *H. M. Westergaard*	Apr., 1942	
Discussion	Sept., Oct., Nov., 1942	Jan. 1, 1943
Moment Balance: A Self-Checking Analysis of Rigidly Jointed Frames. *R. J. Cornish*	May, 1942	
Discussion	May, June, Sept., Oct., Nov., Dec., 1942	Jan. 1, 1943
Mechanics of Creep for Structural Analysis. *Joseph Marin*	May, 1942	
Discussion	Sept., 1942	Jan. 1, 1943
Classification of Irrigable Lands. *W. W. Johnston*	May, 1942	
Discussion	May, Oct., 1942	Feb. 1, 1943
Numerical Procedure for Computing Deflections, Moments, and Buckling Loads. *N. M. Newmark*	May, 1942	
Discussion	May, June, Sept., Oct., Nov., 1942	Feb. 1, 1943
Relative Angular, Linear, and Traverse Accuracies in City Surveys. *S. A. Bauer*	May, 1942	
Discussion	Oct., Nov., 1942	Feb. 1, 1943
Physical Properties of Driven and Undriven Rivets of High-Strength Structural Steels: Progress Report of the Committee of the Structural Division on Structural Alloys	May, 1942	Feb. 1, 1943
Earth Pressure and Shearing Resistance of Plastic Clay: A Symposium	June, 1942	
Discussion	Sept., Dec., 1942	Feb. 1, 1943
Protective and Remedial Measures for Sanitary and Public Health Engineering Services: Second Progress Report of the Sanitary and Public Health Engineering Division of the National Committee of the Society, on Civilian Protection in War Time	June, 1942	
Advancement of Sanitary Engineering: Report of the Committee of the Sanitary Engineering Division on Advancement of Sanitary Engineering	June, 1942	
Statistical Analysis in Hydrology. *L. R. Beard*	Sept., 1942	
Discussion	Oct., Nov., Dec., 1942	Feb. 1, 1943
Permeability of Mud Mountain Dam Core Material. *Allen S. Cary, Boyd H. Walter and Howard T. Harstad*	Sept., 1942	
Discussion	Dec., 1942	Feb. 1, 1943
Entrainment of Air in Flowing Water: A Symposium	Sept., 1942	
Discussion	Nov., 1942	Feb. 1, 1943
Conformity Between Model and Prototype: A Symposium	Oct., 1942	
Discussion	Dec., 1942	Mar. 1, 1943
Distribution Graphs of Suspended-Matter Concentration. *Joe W. Johnson*	Oct., 1942	Mar. 1, 1943
The Hydraulic Jump in Sloping Channels. *Carl E. Kindsvater*	Nov., 1942	Apr. 1, 1943
Application of Soil Mechanics in Designing Building Foundations. *A. Casagrande and R. E. Fadum*	Nov., 1942	Apr. 1, 1943
Flow Characteristics at Rectangular Open-Channel Junctions. *Edward H. Taylor*	Nov., 1942	Apr. 1, 1943
Relation of Undisturbed Sampling to Laboratory Testing. *P. C. Rutledge*	Nov., 1942	
Discussion	Dec., 1942	Apr. 1, 1943
Advances in Sewage Treatment and Present Status of the Art: First Progress Report of the Committee of the Sanitary Engineering Division on Sewerage and Sewage Treatment	Nov., 1942	
Sewer Rental Laws and Procedure: Third Progress Report of the Committee of the Sanitary Engineering Division	Nov., 1942	
Organizing and Financing Sewage Treatment Projects. *Samuel A. Greeley*	Dec., 1942	
Discussion	Dec., 1942	May 1, 1943
Pendleton Levee Failure. *Kenneth E. Fields and William L. Wells*	Dec., 1942	
Discussion	Dec., 1942	May 1, 1943

NOTE.—The closing dates herein published are final except when names of prospective discussers are registered for special extension of time.

* Publication of closing discussion pending.

CONTENTS FOR DECEMBER, 1942

PAPERS

DISCUSSIONS

CONTENTS FOR DECEMBER, 1942 (*Continued*)

INDEXES

For Index to all Papers, the discussion of which is current in PROCEEDINGS, *see page 2*

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

CONCRETE RESERVOIRS OF THE VERTICAL-BEAM TYPE

By C. Maxwell Stanley, Jr.,[1] M. Am. Soc. C. E.

Synopsis

Covered concrete storage reservoirs can be designed by a "vertical-beam" method that offers advantages over conventional types. The method, which is presented herein, utilizes side-walls constructed as simple vertical beams with supports arranged at top and bottom to resist the internal horizontal pressure. The designation "vertical beam" is taken from this wall construction.

With such a design, all of the concrete in contact with the contained water is in compression. Simple methods are afforded for minimizing temperature and shrinkage stresses. These features reduce the formation of cracks on the interior face of the structure and thus, by preventing seepage into the wall, increase the resistance of the structure to deterioration. These advantages are obtained without increase in construction cost over conventional types.

Design Problem

Reservoir Experience.—The design of concrete storage reservoirs is difficult, particularly in climates having severe winters. Many cases of unfortunate experiences with concrete storage reservoirs have resulted from a lack of appreciation of the fundamental design principles.

The designer must obtain a watertight, structurally safe reservoir at a minimum cost. The wall must act both as a membrane preventing the flow of water from the reservoir and as a structural member resisting internal pressure. To fulfil the first function, the walls must be impervious and free from cracks. As it fulfils the second, it must deform to resist stresses and, as it deforms, it will crack wherever appreciable tensile stresses occur. Even though these cracks do not result in objectionable leaks, they permit saturation of the concrete which may lead to rapid damage from freezing and thawing. Even the use of low steel stresses will not eliminate cracks entirely. If the designer can select

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1943.**

[1] Cons. Engr. (Stanley Engineering Company), Muscatine, Iowa.

the design in which the concrete surface presented to the water remains in compression and is free from uncontrolled cracks, he will have done much toward the elimination of seepage into the concrete, with its ensuing deterioration.

Criteria of Wall Design.—The following are listed as criteria of an ideal wall design that will minimize and control cracks:

(1) Concrete in contact with water must be in compression;
(2) Walls must be poured in sections sufficiently small to avoid excessive shrinkage cracks;
(3) The entire structure must be so designed that expansion and contraction may be controlled to avoid cracking of surfaces in contact with water; and
(4) Construction joints must be kept to a minimum, and every construction joint must be treated as an expansion joint.

The failure of conventional types of reservoirs to measure up to these criteria led to the development of the "vertical-beam" design.

Vertical-Beam Type

General Description.—As the name implies, the side-wall (see Fig. 1) acts as a simple vertical beam resisting the internal water pressure, with the horizontal reactions taken by suitable supports. The bottom of the beam sets in a recess

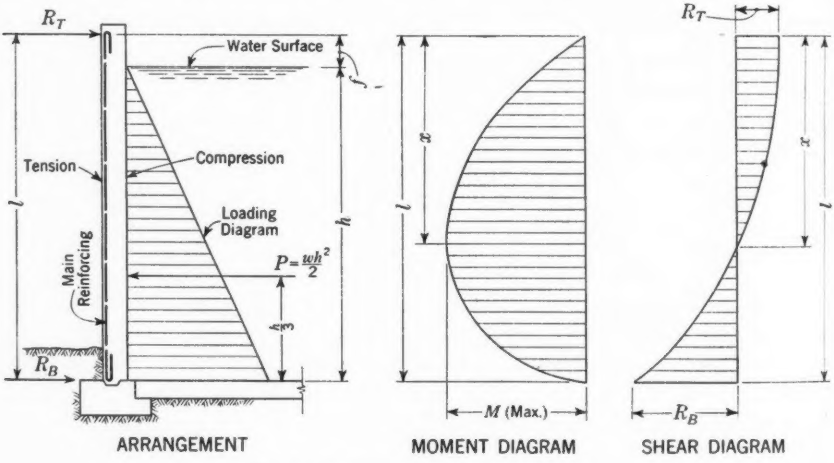


FIG. 1.—TYPICAL VERTICAL-BEAM SECTION

in the footing which is designed so that the wall may rock slightly with changes in temperature and loading, and thus avoid rigidity.

The horizontal reaction at the bottom of the wall is carried into the footing, which is secured against horizontal movement by any one of several methods. At the top of the wall, the horizontal reaction is balanced against an equal reaction at the opposite wall, using the roof slab or rods as ties between the two.

Sections of the side-wall are constructed in one pour from the footing to the top, thus eliminating all horizontal construction joints. The length of wall

sections between vertical joints is kept reasonably short to limit the size of pour and to avoid extreme shrinkage or temperature stresses. All vertical construction joints are treated as expansion joints and are carefully sealed against leakage.

The main steel reinforcement is placed vertically near the outer face of the wall. Temperature steel (not shown in Fig. 1) may be placed horizontally near the outer face and vertically and horizontally near the inner face. The steel from the footing into the beam (if used) is placed in the plane of the main reinforcing near the outer face of the wall.

This design affords a construction in which cracking is minimized by placing in compression all concrete in contact with the water and by carefully providing for temperature and shrinkage stresses. The stresses in the wall may be computed readily by simple structural analysis that need not be outlined or discussed herein.

Support of Top.—Horizontal support of the top of the vertical beam relies upon the symmetry of the structure, which permits the balancing of horizontal forces on opposite sides of the reservoir as shown in Fig. 2. The forces F and F'

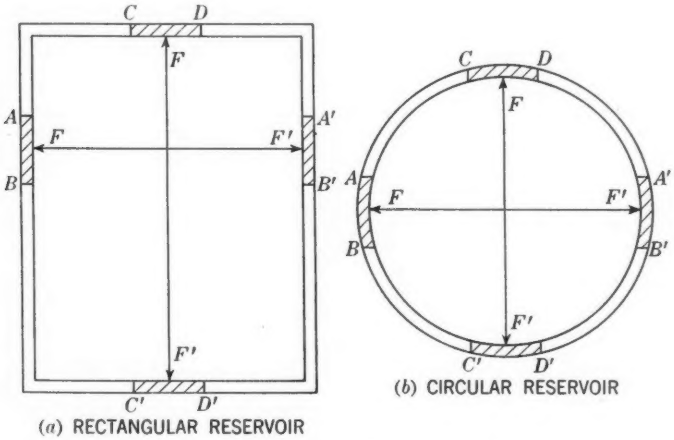


Fig. 2.—Vertical-Beam Walls Arranged to Balance Opposing Forces

will be equal in amount and opposite in direction whether the reservoir is circular, square, or rectangular. The two forces may be balanced by providing a suitable connecting member between the opposite walls. Three different methods are shown in Fig. 3(*a*). The top support will move, due to the expansion or contraction of the tie member and to the strains resulting from the internal load. This movement is taken care of by permitting the vertical beam to rock upon its base.

Support of Bottom.—The bottom of the vertical beam may be supported either by balancing opposing forces, as described for the top of the beam, or by utilizing a horizontal force resulting from the reaction of the wall on the earth or rock or from friction on the base.

Four different arrangements for supporting the bottom of the beam are shown in Fig. 3(*b*). In each of these designs the bottom of the wall is detailed so that it will be free to rock about its base. If tie rods are used between the footing and the wall, they are located in the plane of the main steel reinforcement near the outer wall to avoid tension at the bottom of the slab. A seal

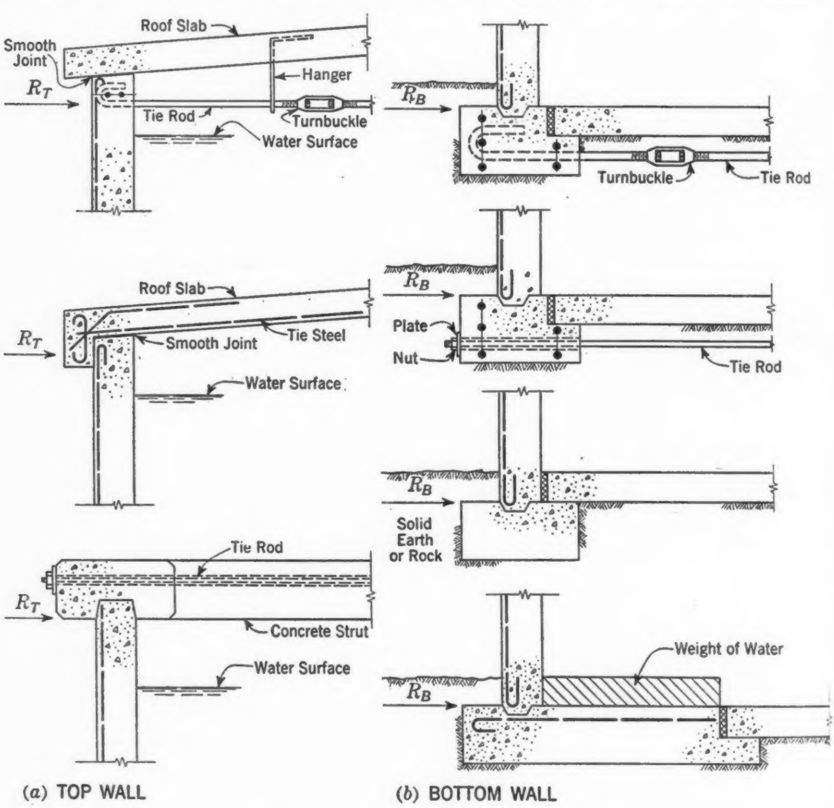


FIG. 3.—METHODS OF SUPPORTING THE WALLS

(not shown in Fig. 3(*b*)) must be provided to prevent leakage through the horizontal joints where the wall joins the footing. All of these arrangements avoid tension in the surface of the wall that is exposed to the water.

Provision of Expansion and Contraction.—Expansion and contraction are taken care of by limiting the dimensions of each pour and by designing the structure with a flexibility that will permit movement of the walls without damage to the structure.

Horizontal construction joints are eliminated and each vertical wall section is limited to such length as will avoid shrinkage or contraction cracks. In the original designs, sections of 30 ft to 40 ft were used, in keeping with customary practice in other concrete structures. However, vertical cracks have developed

in sections of this length and are apparently the result of continued contraction. Therefore, more recent designs have used shorter lengths of wall (less than 20 ft) to eliminate such cracking. Temperature steel may be used as required in the faces of the wall, but should not extend through the vertical construction joints. Expansion joints are used between adjacent wall sections.

Corners of square or rectangular reservoirs are constructed with expansion joints arranged so that movement, or rocking of the wall, will not be resisted by the corners. Changes in temperature will result in the expansion and contraction of the roof slab or supporting members holding the top of the vertical beam. When this occurs, the vertical beam will rock slightly on its base, without changing the stress relationship within the member, as there is no rigidity in the connections between the wall, footing, and roof. Differences in temperature between the inside and outside of the reservoir will cause differential expansion or contraction of the inner and outer surfaces, and the wall will tend to assume a slightly curved section. However, as the wall is not restrained at either top or bottom, it may be distorted in this manner without harm.

Waterproofing.—Waterproofing is required on the wall at the vertical expansion joints and at the horizontal joints between footing and wall. The walls require a waterproofing treatment to fill the pores and increase imperviousness. Several kinds of waterproofing may be used, but very satisfactory results have been obtained with a cement grout containing iron filings.

The expansion joints are composed of poured or premolded material of suitable thickness and require some type of noncorrosive metal steel. After some unsatisfactory experiences with copper seal of conventional design, an

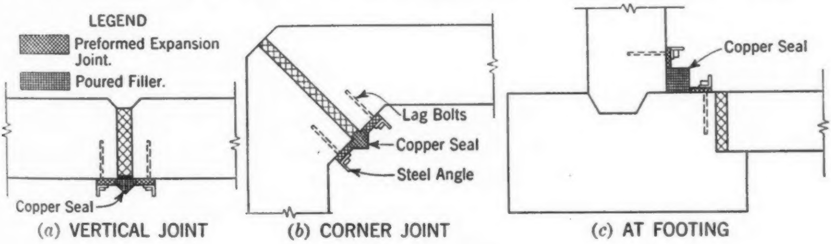


FIG. 4.—TYPICAL SEALS

externally mounted seal has been developed. The seal, shown in Fig. 4(*a*), is installed after the wall is poured and is thus accessible during installation and for maintainance. Its cost is probably a little more than a conventional seal, but any extra cost is fully warranted by its simplicity and dependability (see also Fig. 4(*b*) for a seal at the corner joint).

The joint between the wall and footing also must be made tight, and this can be accomplished either by a modification of Figs. 4(*a*) and 4(*b*) or by the use of a more conventional copper seal (Fig. 4(*c*)). If the head of water within the reservoir is small, the copper seal may be omitted and the joint can be sealed with bituminous material.

Backfilling.—Although the vertical beam design is intended primarily for reservoirs located above ground, the side-walls may be partly backfilled without

any modification of the design. Under such conditions, the simple beam acts as a cantilever with the main reinforcement carrying the tension and with the inner face of the wall remaining in compression. Dowels must then be used near the outside face to tie the wall to the footing; and, at the top of the wall, a type of support must be used in which the wall is free to move inward. A considerable depth of backfill may be placed without requiring any increase in wall section. The footing must be of adequate design to prevent the wall from overturning.

Experience with Vertical-Beam Reservoir

Actual construction and use are the final test of any design. A critical examination of experiences with the vertical-beam design reveals its advantages as well as the problems that have arisen. Both difficulties and advantages call attention to needed changes in details of design and construction. Data on three such reservoirs that have been constructed and used are shown in Table 1.

TABLE 1.—Data on Vertical-Beam Reservoirs

No.[a]	Year built	Capacity (gallons)	Average Dimensions (Feet)			
			Plan	Water depth	Wall height	Spacing[b]
1	1935	3,000,000	122 by 90[c]	18	20	40[d]
2	1939	120,000	41 by 41	9	10	41
3	1941	100,000	34 by 34	9.5	10.5	17

[a] Corresponding to project numbers in Fig. 5. [b] Spacing of expansion joints. [c] Each of two compartments. [d] Joint spacing 30 ft to 40 ft.

Typical details of wall construction and wall support for these three reservoirs are shown in Fig. 5. Experiences during the construction of the reservoirs, and observations after they had been placed in service, have disclosed defects that have been remedied in subsequent designs.

The arrangement of tie rods and struts in the first project (Fig. 5(*a*)) was very effective in preventing the movement of footings, but it was costly and was slow to construct. Even before the job was completed, the simpler arrangements for bottom support shown in Figs. 5(*b*) and 5(*c*) were developed for use in subsequent designs. These arrangements permit the tightening of tie rods after all other construction is complete.

The method of top wall support in the first project (slab rigidly connected to wall) hindered the free action of the wall. Cracks developed immediately over each vertical wall joint and extended back into the roof slab. Also, diagonal cracks developed across each of the roof slabs some 8 ft or 10 ft back from the intersection of the outside wall. This crack was matched by vertical cracks in each of the walls a like distance from the corner. All these cracks seemed to be the direct result of restricting the free action of the wall and roof by a rigid interconnection. This led to the development of other methods of top support (see Fig. 3(*a*)) that permitted free action of roof and wall, and these methods were adopted in subsequent designs.

Trouble with the expansion joints used on the first project developed when the reservoir was filled. Efforts to repair them were generally unsatisfactory until a seal similar to that of Fig. 4(*a*) was used. This type of seal has been so

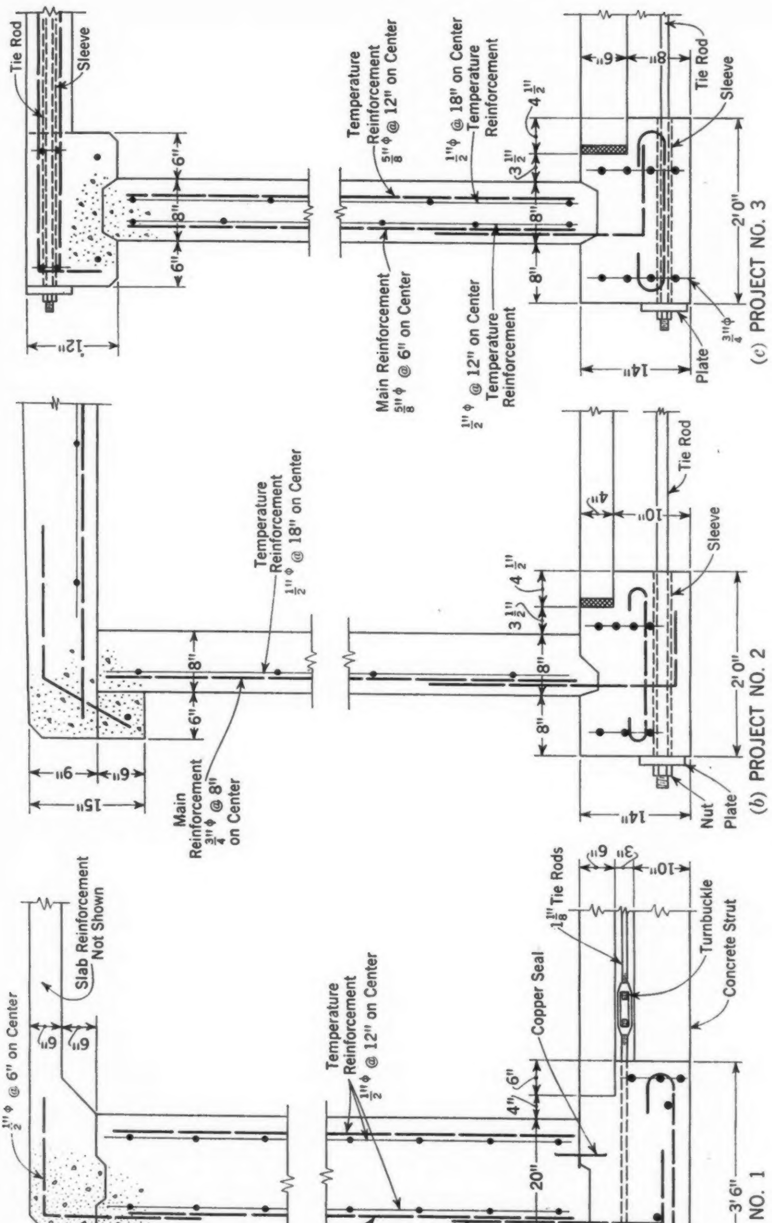


FIG. 5.—TYPICAL DETAILS OF SIDE-WALLS

effective that it has been generally applied not only on successive reservoir designs, but on other hydraulic structures.

The most serious problem encountered in the first designs was the formation of vertical cracks between construction joints. On the first structure (Fig. 5(*a*)), these cracks were not apparent when the structure was placed in service, but have developed gradually over the years. On the second reservoir (with lighter walls), the cracks were observed before construction was completed (see Fig. 5(*b*)). Their location and development indicate that they are the result of contraction and shrinkage stresses and that the 30-ft to 40-ft intervals between expansion joints are too large for the differential temperature stresses which occur in an exposed reservoir wall. The last design (Fig. 5(*c*)) used a much smaller interval between joints, and no vertical cracks have been observed.

The aforementioned troubles, although annoying, have not been serious and are probably no more than may be expected with any new design. The reservoirs that have been constructed have demonstrated the soundness of the theory of design. The construction is simple, and costs are low. The walls have acted as planned and have rocked with the expansion of the roof. The corner and intermediate vertical joints have reacted as expected. The reservoirs are giving satisfactory service and the troubles encountered have only served to improve subsequent design. The same principle of vertical-beam construction has been applied to several other structures.

Conventional Designs

The more common designs that have been applied to storage reservoirs include the following:

(1) Circular walls acting as shells to resist internal bursting pressure.
- (*a*) Conventional type with tension steel in concrete.
- (*b*) A pre-stressed reinforcement type developed by William S. Hewett, with tension steel outside the concrete.

(2) Retaining walls utilizing the weight of the impounded water to resist the outward pressure.
- (*a*) Cantilever.
- (*b*) Buttress or counterfort.

A description of each of these types and a statement of their obvious advantages and disadvantages follow.

Circular Walls.—Both the conventional and pre-stressed type of circular walls are based on the same fundamental theory. The walls are designed as shells to resist the bursting pressure of the water within the reservoir. The tension at any point on the wall varies with the diameter of the reservoir and the depth of the water. As concrete cannot resist tensile stress of substantial magnitude, circular hoops of steel must be used. The area of steel required will vary directly with the depth. In the conventional design, the steel is embedded within the concrete wall, whereas in the pre-stressed design the steel is placed around the wall and is pre-stressed.

The placing of the steel within the wall in the conventional design creates a serious problem, as the forces acting on the wall tend to increase its circumference, and the concrete, having limited elasticity, is cracked. Nothing can be done to prevent this cracking although the size of cracks may be controlled. Attempts often are made to avoid cracking by limiting the stress in the steel to about 9,000 or 10,000 lb per sq in., but even under these conditions some cracking is likely to occur. Furthermore, other cracks result from horizontal and vertical construction joints and from temperature and shrinkage stresses.

The disadvantages of the conventional circular reservoir are superbly overcome in the pre-stressed type, which has been quite widely used in recent years since this design separates the membrane from the structural member. This is accomplished by placing the hoop steel outside an inner membrane composed of narrow, vertical, concrete slabs without horizontal joints. The slabs are pulled together by tightening turnbuckles in the circular hoops, thus pre-stressing both slabs and hoops to such a point that, even with the reservoir filled, the slabs are still in compression. This design, with its pre-stressing, readily cares for shrinkage and temperature stresses and achieves the goal of all reservoir design—an inner concrete membrane placed in compression and free from cracking. Its only disadvantages are its mandatory circular shape, and the relatively high cost for certain ratios of depth and diameter.

Retaining Wall Types.—Wall designs of the retaining wall type depend primarily on the weight of the water within the reservoir to offset the overturning moments exerted by the water pressure on the side-wall. Either the cantilever or buttress type may be used. With the cantilever type, the entire inner face of the vertical wall is placed in tension, together with the top surface of the footing within the reservoir. With either buttresses or counterforts, the vertical wall and the base slab are designed as horizontal beams and are usually constructed continuously past two or more supports, thus placing a part of the inner face of the vertical wall and the top face of the footing in tension.

Thus, when the principles of a retaining wall are used with either type of design, a part, or all, of the concrete in contact with the water will be in tension. Such design also requires a rigid structure that does not lend itself readily to the elimination of horizontal construction joints, or to the minimizing of cracking due to temperature or shrinkage stresses.

Appraisal of Vertical-Beam Design

The vertical-beam method of reservoir construction has been developed and tested so that its advantages and disadvantages can be established reasonably well. Reservoirs may be designed successfully, using the vertical-beam method, provided that the details which have given trouble are avoided. The following conclusions may be drawn:

(1) The basic theory of the "vertical-beam" reservoir is sound. The tendency for concrete to crack is minimized by placing all wet surfaces in compression and by providing sufficient flexibility in the structure to reduce the tendency toward cracking from temperature movements and stresses. None of the conventional designs (except the pre-stressed concrete type) accomplish this.

(2) The vertical-beam type of reservoir offers extreme simplicity of both design and construction. Structural analysis is direct and not complicated. Construction is concerned only with plain sections and surfaces.

(3) The construction costs of vertical-beam reservoirs compare favorably with the conventional types. The flexibility of the vertical-beam type of construction and the absence of rigid connections permit the efficient use of concrete and steel and allow the selection of relatively thin sections. These facts, together with the simplicity of construction, permit economy.

(4) The design is most aptly suited to rectangular, or square, reservoirs. Although it may be applied to circular reservoirs, the latter must be of large diameter and the vertical sections must be made sufficiently short so that horizontal bending stresses will not be developed in the curved sections.

(5) The design has been limited to depths such that the vertical walls can be poured in one lift. It may be possible to use horizontal construction joints with suitable seals, but it seems that a better design will be obtained if this is avoided.

(6) This type of construction is limited as to the amount of backfill that may be used on the outside. Under backfill conditions, the vertical wall acts as a cantilever, and its strength as such is limited.

(7) This type of design is also probably limited to those cases where good foundations are available. If a reservoir must be constructed upon soft foundations, a continuous base slab probably will be dictated.

Acknowledgments

The writer wishes to acknowledge the suggestions that have been made by Arthur E. Stanley, Assoc. M. Am. Soc. C. E., and by Herman S. Smith and Allen H. Dunton, Juniors, Am. Soc. C. E., who have been associated with him in the preparation of the detailed designs of these reservoirs.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
PAPERS

ILLINOIS WATER LITIGATION, 1940-1941

By Langdon Pearse,[1] M. Am. Soc. C. E.

Synopsis

In 1940–1941, hearings followed a petition to the U. S. Supreme Court by the State of Illinois for a temporary increase in diversion from Lake Michigan. The inquiry was directed to ascertain the effect of odors from a large pool in the heart of Joliet, Ill., on health, the remedial measures available, and the condition of the sewage treatment works of The Sanitary District of Chicago. The Special Master held that no menace to health was proved and that no remedial measures were available as effective or as promptly as the proposed completion of the sewage treatment works of the Sanitary District. The U. S. Supreme Court denied the petition.

General

Considerable interest was aroused by the petition of the State of Illinois, dated January 11, 1940, for a temporary relaxation of the diversion order restricting the flow of water from Lake Michigan to relieve the condition existing in the upper Illinois Waterway, particularly at Joliet (see Fig. 1). The litigation was somewhat unique in that it involved the question: "When is pollution a menace to health" rather than the well-defined problem of the abatement of a nuisance. Curiously the attorneys on both sides admitted that no cases could be cited as legal precedent.

Prior to July 1, 1930, the War Department Permit of March 3, 1925, limited the diversion to an annual average of 8,500 cu ft per sec, with an instantaneous maximum of 11,000 cu ft per sec. This permit expired on December 31, 1929. A new permit was then issued authorizing an annual average diversion of 7,250 cu ft per sec, or such lesser annual average diversion as would restrict the average annual flow measured at Lockport, Ill., to 8,500 cu ft per sec, until July 1, 1930. The actual diversion in 1929 was 7,699 cu ft per sec, and in 1930, 6,658 cu ft per sec.

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1943.**

[1] San. Engr., The San. Dist. of Chicago, Chicago, Ill.

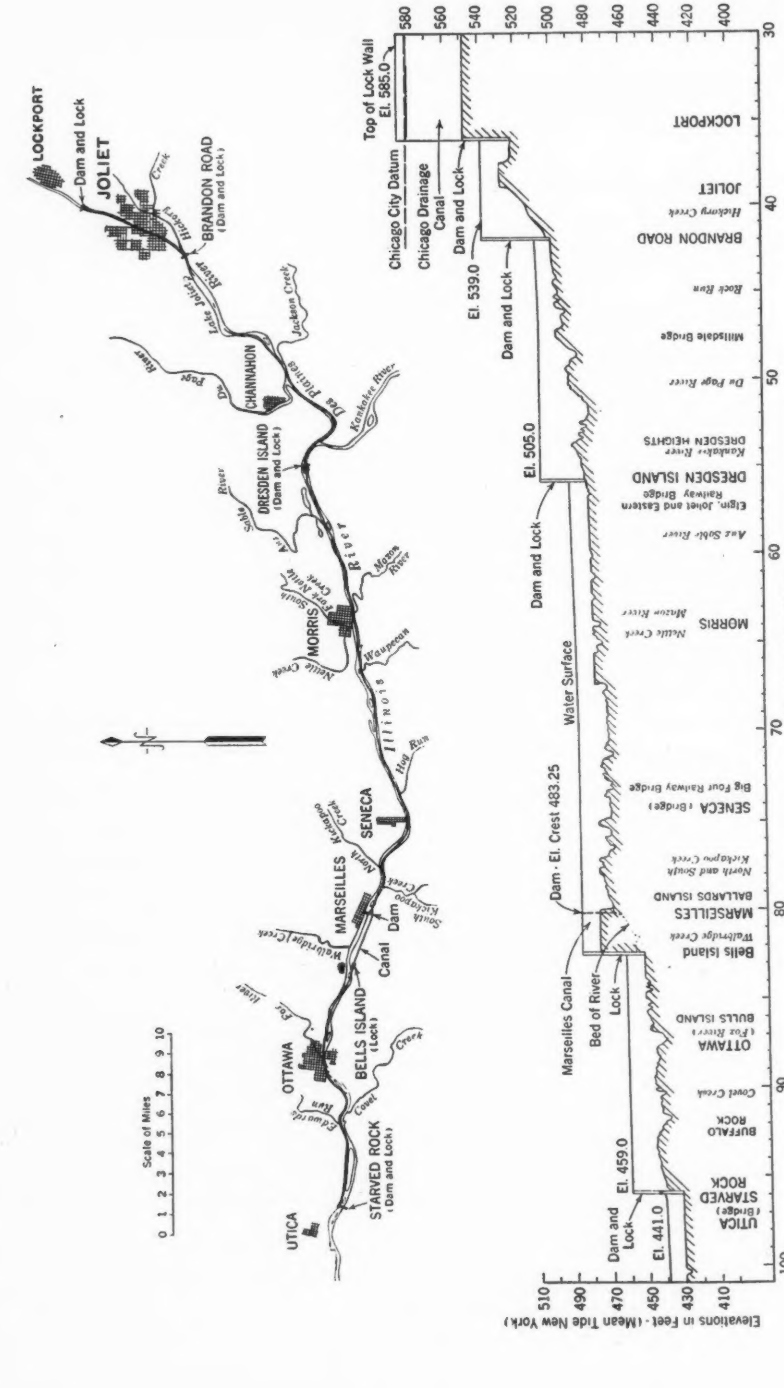


FIG. 1.—GENERAL PLAN AND PROFILE, ILLINOIS WATERWAY

This litigation was a result of the decree of the Supreme Court of the United States issued April 21, 1930, which provided that the annual average diversion at Chicago, Ill., from Lake Michigan should be reduced in three steps:

Step	Date	Total diversion (cu ft per sec)
1	July 1, 1930	6,500
2	December 31, 1935	5,000
3	December 31, 1938	1,500

These allowances are in addition to the domestic pumpage of Chicago, which amounts to approximately 1,700 cu ft per sec. A controlling works was also to be built at the mouth of the Chicago River on or before December 31, 1935. The diversions were made on schedule and the controlling works were built.

The 1930 decree simply limits the flow of water. It does not specify the type of sewage treatment works to be built or when they shall be completed, although this was discussed by the Special Master, the Hon. Charles E. Hughes, in his report on Re-Reference.

The Court did not foresee the depression which retarded tax collections in 1931–2–3 and thereby stopped The Sanitary District of Chicago from selling bonds because of the technical default of its then outstanding issues. Consequently the timing of the completion of the sewage treatment works gradually fell out of step with the reduced diversion after 1935, with resulting difficulty in 1939, after the final reduction was made. The Sanitary District took the position that it should follow the decree to the letter. After the first two steps, the sewage treatment works still provided enough cushion to show some improvement in the condition of the upper Illinois Waterway until 1938, when the third step reduced the flow so greatly that conditions were created that were worse than at any time in the preceding 30 years.

The late Chief Justice William Howard Taft had instructed the Special Master to determine the diversion required for the Port of Chicago. This was interpreted to include the Main Channel at Lockport. Thus, by the Taft instructions, conditions below Lockport were ignored in preparing the Report on Re-Reference and the 1930 decree, because, when Mr. Taft was Chief Justice, the Des Plaines River from Lockport to its junction with the Kankakee River, and thence the Illinois River to Utica, Ill., were not navigable waters of the United States. By the River and Harbor Act of July 3, 1930, the entire waterway from the Chicago River to Utica became a navigable waterway of the United States, more than two months after the decree.

The "Illinois Waterway" is the term now applied to the waterway from Lake Michigan to the Mississippi River. In July, 1930, the United States took over the upper waterway from Utica to Lockport, then under construction by the State of Illinois, and completed the work (7).[2] In the spring of 1933, the waterway was opened for through navigation. In this plan of slack-water navigation a number of pools were created by the dams at Brandon Road, Dresden Island, Marseilles, Ottawa, Peoria, and La Grange, all in Illinois. This paper is concerned principally with the happenings in the upper pool,

[2] Numerals in parentheses, thus: (7), refer to corresponding items in the Bibliography, in the Appendix.

known as the Brandon Pool (Fig. 1), which extended from Lockport to the Brandon Road dam, a distance of about 4.84 miles, through the heart of Joliet, a city of 42,365 population (1940 census). The pool varies in width from 200 ft to 1,400 ft, with a maximum depth of 27 ft. Prior to the closing of the Brandon Road dam in 1933, the flow passed through Joliet down river with considerable rapidity, and consequently with a high rate of re-aeration. To the end of 1938, however, little was heard of conditions in the pool, although sludge was accumulating in it.

With the reduced diversion, in 1939, conditions changed rapidly for the worse. The 5-day biochemical oxygen demand (B.O.D.) at Lockport rose rapidly. The dissolved oxygen disappeared not only at Lockport (Table 1), but upstream for 30 miles into the South Branch of the Chicago River at Damen Avenue (Table 2). The conditions along the Main Channel were the worst known for more than 30 years.

TABLE 1.—DISSOLVED OXYGEN AND BIOCHEMICAL OXYGEN DEMAND AND DISCHARGE AT LOCKPORT, ILL.

Month	1938				1939				1940			
	Discharge (cu ft per sec)	Temperature (degrees C)	PPM		Discharge (cu ft per sec)	Temperature (degrees C)	PPM		Discharge (cu ft per sec)	Temperature (degrees C)	PPM	
			Dissolved oxygen	5-Day B.O.D.			Dissolved oxygen	5-Day B.O.D.			Dissolved oxygen	5-Day B.O.D.
Jan.	6,408	1.5	7.4	22.6	2,911	5.0	0.5	28.5	2,949	1.5	1.8	14.8
Feb.	7,422	3.5	7.0	21.0	3,989	3.5	2.2	26.6	2,802	5.0	0.4	23.0
Mar.	7,607	6.0	5.6	19.4	3,200	6.5	0.8	25.0	3,167	5.5	1.0	23.2
Apr.	7,686	10.5	3.2	16.3	2,718	13.5	0.1	31.4	3,026	10.0	0.2	26.6
May	6,311	15.0	0.5	17.6	2,623	17.5	0.0	42.3	3,344	14.5	0.1	25.3
June	6,701	19.0	0.3	18.8	4,227	21.5	0.0	37.8	2,876	21.0	0.0	27.9
July	6,814	23.0	0.1	18.5	2,880	24.5	0.0	26.5	3,609	23.5	0.0	18.7
Aug.	6,740	23.0	0.1	14.6	2,902	25.0	0.0	26.2	3,923	23.0	0.1	15.2
Sept.	7,247	20.5	0.3	13.4	2,826	24.0	0.0	23.0	3,125	22.0	0.1	14.3
Oct.	5,512	17.0	0.2	18.6	3,022	18.0	0.0	18.1	3,186	18.5	0.2	14.1
Nov.	5,861	10.5	1.2	18.5	2,820	11.5	0.4	13.8	2,824	11.0	0.8	13.6
Dec.	5,469	4.5	4.3	21.8	3,469	8.0	1.3	13.2	4,996	4.0	4.9	24.7

TABLE 2.—COMPARISON OF CONDITIONS IN AUGUST; MAIN CHANNEL AND SOUTH BRANCH OF THE CHICAGO RIVER

(Units Are Parts per Million)

Description	Chicago Avenue		Damen Avenue		Cicero Avenue		Summit	
	1938	1939	1938	1939	1938	1939	1938	1939
Dissolved oxygen	2.2	0.6	5.5	0.0	2.7	0.0	1.1	0.0
Five-day B.O.D.	6.3	8.2	6.8	32.7	8.3	33.0	13.5	34.2

In the summer of 1939, the outcry from Joliet became so great that on January 11, 1940, the State of Illinois applied for a modification of the 1930 decree to permit a temporary increase of the diversion to 5,000 cu ft per sec in addition to domestic pumpage, until December 31, 1942. After a hearing

on this application, the Court, on April 30, 1940, ordered that the petition of the State of Illinois and the return of the Complainant States be referred to Monte M. Lemann as a Special Master

> "* * * with directions and authority to make summary inquiry and to report to this Court with all convenient speed with respect to the actual condition of the Illinois Waterway by reason of the introduction of untreated sewage, and whether, and to what extent, if any, that condition constitutes an actual menace to the health of the inhabitants of the Complaining Communities, and also with respect to the feasibility of remedial or ameliorating measures available to the State of Illinois without an increase in the diversion of water from Lake Michigan."

(In this paper the petitioner, the State of Illinois, is referred to as "Illinois"; the Complainant States, Michigan, Minnesota, New York, Ohio, Pennsylvania, and Wisconsin, are referred to as "Opposing States"; and the Special Master, Mr. Lemann, is referred to as the "Master."

The hearings were held in Chicago over a period of months. The case was argued in New Orleans, La., in February, 1941, before the Master, who rendered a report to the Court on March 31, 1941. On May 26, 1941, the Court denied the petition.

The original petition (January, 1940) requested a temporary increase in diversion to 5,000 cu ft per sec until December 31, 1942. After the completion of the hearings the petition was modified. In its revised petition, Illinois requested a temporary increased diversion during the warmer months of 1941 and 1942 sufficient to maintain a minimum of 1 ppm of dissolved oxygen in the Brandon Pool. The expected need for increased diversion was set up as follows:

Month	1941	1942
April	1,000	....
May	3,000	2,000
June	5,000	4,000
July	6,000	5,000
August	5,000	3,000
September	3,000	1,000
October	1,000	....
Annual average increase	2,000	1,250

The annual average increase in diversion thus requested was 2,000 cu ft per sec in 1941 and 1,250 cu ft per sec in 1942. The earlier records in the case and the findings of Special Master Hughes indicate that the lowering effect of such temporary diversions would not be more than ½ in. in 1941 and ⅓ in. in 1942 in Lake Michigan, Lake Huron, and Lake Erie.

Minimum Dissolved Oxygen

F. W. Mohlman and the writer testified that in the absence of sludge deposits at least 1 ppm of dissolved oxygen is necessary to prevent nuisance. Referring to New York vs. New Jersey, 256 U. S. 296, 311, decided in 1921, the Master expressed his belief that the weight of opinion is that the presence of any dissolved oxygen is sufficient. In that case, however, the Court held that

25% to 50% of saturation is the dissolved oxygen necessary to prevent offensive odors from decomposition of organic matter deposited in the body of water under consideration. As Lake Michigan contains about 8 ppm dissolved oxygen in the summer months, this yardstick would require 2 to 4 ppm of dissolved oxygen in the warmer months in Brandon Pool.

The evidence shows that the most important factor responsible for the foul conditions of the Main Channel and Brandon Pool is undoubtedly the deposits of putrefactive sludge that formed since the diversion was reduced on January 1, 1939. In Brandon Pool, this deposit has accumulated since 1933, from residual solids in the effluents of the various works of the Sanitary District, from raw solids, the discharge of sludge at Stickney (the site of the West and Southwest works of the Sanitary District), and from the storm-water solids.

The accretions of prior years and the preceding winter, which lay dormant through cold weather, exerted a potent effect with the onset of warmer weather and a water temperature of 62° F or higher.

The engineering witnesses on both sides of the case agreed that such sludge deposits are potent in their avid demand for oxygen, and are the most important factor in the production of the foul condition in the Main Channel and Brandon Pool. All agreed that the discharge of settling suspended matter, whether in sewage partly treated or as sludge, should be stopped as soon as possible. This is what the Sanitary District program contemplates doing. Thus the Opposing States and Illinois were agreed that the solids must be kept out of the channels of the Sanitary District and that the sewage must be treated to as high a degree as practicable, as quickly as possible.

The conditions in the Main Channel and Brandon Pool during 1939 and 1940 continued through 1941 and 1942 and will continue thereafter although on a gradually diminishing scale, until the sewage treatment works of the Sanitary District are completed.

The sludge deposits require a long time to lose their putrefactive character or potency. Mr. Mohlman testified that a test of the oxygen demand of sewage sludge overlaid by Illinois River water showed consumption of oxygen for more than 1,100 days. For the first 400 days, this was confirmed by W. Rudolfs, M. Am. Soc. C. E. (1). The results show that during the first summer, for a period of 120 days, the loss of oxygen demand may range from 50% to 55%. During the winter the sludge lies dormant. The following spring possibly 20% to 25% more of the B.O.D. is destroyed. Reference was made to studies of Karl Imhoff, M. Am. Soc. C. E. (2), and the report of the Committee on Sewage Disposal of the Engineering Board of Review (3) which found that the sludge deposits in the Upper Illinois River practically doubled the effect of pollution during the summer.

The condition of the waterway, from Damen Avenue in Chicago to Brandon Road Dam below Joliet, was bad in 1939 and somewhat better in 1940, because of the increased treatment due to the starting of the Southwest works in the summer of 1939. By April, 1940, all the sewage in the Sanitary District was picked up for treatment in its plants except for about 70 mgd and by October, 1940, for about 35 mgd.

From testimony of laymen and experts, there is no question but that hydrogen sulfide was set free. There was controversy over the concentration in the atmosphere, the possible zone of effect, and the presence of other ingredients such as mercaptan, indol, skatole, and cadaverin. One expert witness, C. W. Muehlberger, indicated that persons might become accustomed to hydrogen sulfide but not to fecal odors.

The Health Problem

The proceedings before the Master were clearly divided into three parts, considering—

1. Complaints and the health conditions along the Waterway;
2. Status of sewage treatment by the Sanitary District and the outlook for the summers of 1941 and 1942; and
3. Available remedial or ameliorating measures.

Illinois contended that the present condition of the Illinois Waterway in the Joliet-Lockport area constitutes not only an actual menace to health but also a definite injury to health. Thus, whether or not the Waterway caused the effects alleged to the health of the inhabitants is the initial question in determining whether the conditions, as disclosed by the proof, constitute an actual menace within the meaning of the Court. The evidence shows a two-fold health problem:

(*a*) The actual detriment to health caused by the inhalation of gaseous odors emanating from the Waterway; and

(*b*) The potential hazard, danger, or menace to health occasioned by the present highly polluted and foul condition of the Waterway.

The Opposing States contended

> "* * * that the phrase 'actual menace to health' as used in the order of the United States Supreme Court means that the effects on the health of the people residing near the Waterway must be real and exist in fact, as opposed to any potential, speculative, hypothetical views on what might happen, unsupported by any real cases or actual happenings."

Thus the Opposing States maintained that the evidence falls far short of establishing a menace to health and at most shows only annoyance, discomfort, and inconvenience suffered by some people.

Complaints of Lay Witnesses at Joliet.—The complaints as to the Waterway emanated entirely from residents of Joliet and Lockport, and chiefly from those in Joliet residing alongside the pool and near it. The population of Joliet was reported to be 42,365 in 1940. The Health Commissioner of Joliet estimated 50,000 people were affected, living in and around Joliet. Through Joliet, the Waterway is crossed by eight bridges.

No water from the Waterway is used for drinking purposes. There is no swimming in it. All complaints were based on the offensive odors from the pool, and the effects ascribed to them.

A total of 127 witnesses testified at Joliet, made up as follows:

Lay residents of Joliet	81
Physicians residing in Joliet	15
Nurses and employees of St. Joseph's Hospital	24
Lay residents of Lockport	7
Total	127

It was stipulated that if 200 additional witnesses were called, their testimony would be substantially the same. Three fourths of the witnesses resided within four blocks of the canal, and more than half within two blocks.

The complaints generally were that the pool in the hot summer months of 1939 gave off extremely offensive odors, which in many cases caused nausea, lack of appetite, or inability to sleep. The character of the odors was described variously as the odor of sewers or outhouses or of rotten eggs. Some called it a "stink," others a "terrible" or "unbearable" odor.

All the witnesses agreed that the pool had always given off objectionable odors, but in 1939 the odors were much more offensive than theretofore. The weight of the evidence was that in 1940 the conditions were somewhat better.

The nurses at the hospital (one and one-half to two blocks from the pool) complained of the odors. Many of the patients suffered nervousness, nausea, vomiting, sleeplessness, and headache. Some of the patients were anxious to go home early. However, the average period of stay during 1939 was about one day less than the average for general hospitals in the United States.

The superintendent of grade schools at Lockport (eight blocks from the Waterway) and the principal of an elementary school at Joliet (600 ft from the Waterway) testified as to the type of odor and its nauseating effect on the children in 1939, and to a lesser extent in 1940.

The percentage of sick leaves of the employees in the U. S. Engineer office from Lockport to Starved Rock was somewhat higher in 1939 than in 1938. However, it was but little less than in the Chicago area. In 1940, the percentage in the Joliet area was less than in 1939.

The Health Commissioner of Joliet received hundreds of complaints in June, July, and August, 1939. He stated that there was danger of an epidemic from flies and mosquitoes alighting on the scum; that the stench was nerve wracking; and that the odorous gases irritated the mucous membrane of the nose, throat, and sinus. However, the health conditions in Joliet were good. The only difference in 1939 was the odor.

The other fourteen Joliet physicians considered the pool a menace to health for similar reasons. One was a government surgeon who noted skin diseases attributable to the foul water. Another noted an increase in impetigo along the Waterway from year to year.

The Master concluded that no cases of specific illness clearly attributable to the pool were cited by the physicians.

The statistics on reportable diseases in Illinois show that the case rate per 100,000 inhabitants in 1939 was less in Will County (in which Joliet lies) than the average for the state except for poliomyelitis, malaria, and influenza. In

1939, the death rate was lower in Will County for all reportable diseases than the average for the state. The typhoid rate in Will County in 1939 was 0.9 per 100,000 as compared with an average of 1.8 for the years 1930 to 1938, and a rate of 1.4 for the state. The statistics for the cases and deaths from typhoid show lower rates for counties adjoining the Waterway than for those that do not.

Testimony by Opposing States on Health Conditions.—Casual tests on August 3, 1940, disclosed no trace of hydrogen sulfide in the air over the pool. The bacterial tests on July 16, 1940, disclose only the usual bacteria found in air. Various experts testified that there was no possibility of an epidemic in the adjoining community due to the transmission of germs or bacteria by air from the Waterway. On September 17, 1940, the number of bacteria found over the pool was about one tenth that of other stations. On September 28, 1940, samples of water from the pool showed hydrogen sulfide from 0.1 to 0.2 ppm with an air temperature ranging from 34° to 74° F.

W. H. McNally, M. D., a toxicologist, testified that hydrogen sulfide in the air, concentrated as low as 50 ppm, may produce nausea. A concentration of 100 ppm may cause local irritations and depression of the nervous system. He doubted if such a concentration could occur in the atmosphere over the pool. A concentration of 25 ppm might discolor paint. He quoted Alice Hamilton and Philip Drinker as stating that not more than 20 ppm should be tolerated in industry. The sensitivity of individuals varies. He knew of no recorded instance of damage to health from hydrogen sulfide in a waterway like the Illinois Waterway.

All the physicians, health authorities, and sanitary experts (nine in all) who appeared as witnesses for the Opponents agreed that the Waterway did not constitute a menace to the health of persons living along it.

Testimony for Illinois on Health Conditions.—Mr. Muehlberger, an expert toxicologist, testified that the complaints of nausea, headaches, insomnia, and loss of appetite "may have been caused by hydrogen sulfide." He could not refer to any record in medical literature where an individual was overcome in the open air from a polluted waterway. Upon exposure to more than 20 ppm but less than 50 ppm, the average person would complain of irritation of the mucous surfaces of the eyes, nose, and throat. As stated, decomposing sewage produces organic sulfide compounds, including indol, skatole, mercaptan, and cadavrin, all having offensive odors. Mr. Muehlberger considered the conditions of the Waterway a very definite menace to the health of the inhabitants in the region of Joliet. People may not have died or required hospitalization, but they may have been uncomfortable and unhappy. They are entitled to a decent living condition with good, healthy air. Any contamination of the air is a menace, if a sound state of health is upset. Mr. Muehlberger testified that the odor of the pool in 1939 irritated his throat and nasal passages. However, he had noticed odors in prior years.

A. J. Carlson, M.D., a physiologist, testified that on October 13, 1940, "the present condition is more than a menace to health, it is inimical to health." His conception of a menace to health was typified by conditions which, through

accidents or any ordinary course of human events, can lead to injury to health. He regarded such a polluted pool as a menace even if the water is not used for drinking or cooking or bathing. In his opinion, the fact that the communicable disease rate in Joliet is no higher than in other places is no factor whatever in determining whether the pool is a menace to health. Either the inhabitants of Joliet are lucky or the medical statistics are not what they are supposed to be.

Dr. Carlson stated that the Waterway has been a menace since 1934, but that now the menace has been increased. He used the words "menace to health" in a scientific sense as a theoretical danger which cannot be measured in any statistics or in any outward observation by a physician. The restoration of the diversion to 5,000 cu ft per sec was a lessened menace. A. C. Ivy, M. D., a physiologist, corroborated Dr. Carlson, holding that the disturbances reported due to the odor showed an unhygienic condition working as a detriment to the health of the people so affected. He knew of no recorded case of an individual being overcome by hydrogen sulfide from a waterway. H. A. McGuigan, M. D., a pharmacologist, agreed with Drs. Carlson and Ivy, pointing out that hydrogen sulfide was not the only odor in the air from the Waterway.

General Authorities on Sewage Odors

Reference was made to a report of the American Public Health Association which stated (4):

> "A long prevalent theory that sewage odors are directly responsible for disease has been definitely refuted. It is now realized that the physiological effect, if any, is indirect. Odors, by causing worry, loss of sleep, loss of appetite, etc., may be a contributory cause of ill health, and certainly cause discomfort. The courts, in general, have held that the creators of an odor nuisance are responsible therefor, and as far as municipal sewage disposal is concerned, the odor hazard appears to have been shifted from the public health to the public pocketbook."

M. J. Rosenau, M. D., was quoted (5):

> "While odors may be unpleasant, they are not known to seriously influence health. Contrary to common opinion, they are not by any means a reliable sign of danger. * * *
>
> "The effect of odors upon health is not well understood. * * * Odors influence the nervous system in various ways; some stimulate, others depress."

Condition of Waterway in Earlier Years

In weighing the claims of Illinois that the situation in Joliet and Lockport presents a menace to health which makes imperative an increase in diversion, the Master assembled all the available references to the earlier conditions in the Canal and Des Plaines River, which he cited as evidence of gross pollution. This analysis, however, failed to emphasize the changes brought about by the treatment works of the Sanitary District, and the marked improvement in the upper Waterway to the end of 1938. The conditions he cited as of 1872, 1891, 1911, 1919, 1921, 1922, and 1927 had been radically changed for the better.

Conclusions as to Health Conditions

For many years there have been odors from the Des Plaines River at Lockport and through Joliet. In 1925 when the river flowed over rapids at Joliet, the odors were offensive. From 1930 to 1938 conditions at Joliet improved, notwithstanding the reductions in the flow from Lake Michigan because of the added treatment which the Chicago sewage gradually received. The people at Joliet had become accustomed to this improvement. When the flow from Lake Michigan was reduced on December 30, 1938, conditions retrogressed and complaints ensued from the residents, who compared the 1939 conditions with those prevailing in 1938 and immediately prior to 1938. They had either forgotten the conditions prevailing prior to 1930, or, not unnaturally, felt that the improvement should continue. The nuisance conditions in 1939 were much worse than in 1938. They were not as bad in 1940 as in 1939.

The Master stated that this summary disclosed the facts as he found them. In deciding whether these facts constitute a "menace to health" within the meaning of the instructions of the Court, he indicated he lacked a certain guide. The dictionary definitions of menace he considered of no avail, because the Court itself will determine what it meant.

The Master concluded that the Court did not use the expression "menace to health" in the sense which Drs. Carlson and Ivy ascribed to it. If the Court had so intended, it would not have been necessary to refer the case to a Master, because the facts were clear in the papers in the case. If the test be that urged by Drs. Carlson and Ivy, it is clear that a menace would be found to exist. Accepting this definition, the increase of the flow to 5,000 cu ft per sec would not eliminate the menace. Within the suggested definition the stream would continue a menace as long as any untreated sewage is discharged into it.

The Master then stated that

> "It is obvious that the presence of untreated sewage in an open stream is not in accordance with proper standards of sanitation and should be abated. * * *
>
> "The record leaves no reasonable doubt as to the safety of the water supply of Joliet and Lockport. Nor can the effect of odors upon invalids and persons of less than average health be accepted as any test of health menace; even in cases of private nuisance the test is the effect of the acts complained of upon persons of ordinary sensibilities and in normal health. In the present case it is Illinois itself which is creating the nuisance of which it complains and of which it seeks to be relieved by water which has in effect been adjudged by the Court to belong to the opposing States. * * *
>
> "My conclusion is that the facts proven do not establish any menace to the health of the inhabitants of Joliet and Lockport or elsewhere along the Waterway requiring an increase in diversion in water from Lake Michigan."

Present Status of Sewage Treatment by Sanitary District

The Sanitary District comprises 442 sq miles in area, including Chicago and 59 other municipalities. H. P. Ramey, M. Am. Soc. C. E., testified that this is divided into four main projects, with sewage treatment works as specified:

1. North Side Works.—This is an activated sludge plant, with a present average capacity of 250 mgd. It went into service on October 3, 1928. The effluent goes to the North Shore Channel. The excess sludge and the preliminary settling sludge are pumped 17.5 miles to the Southwest works for ultimate disposal.

TABLE 3.—PROGRESS IN THE TREATMENT OF SEWAGE AT

Year	(a) FLOW (YEARLY AVERAGES, IN MILLION GALLONS DAILY)						(b) RECORD OF SLUDGE AND GRIT,			
	Des Plaines River (1)	Calumet[b] (2)	North Side[c] (3)	Southwest (4)	West Side (5)	Grand total (6)	Des Plaines River (7)	Calumet[b] (8)	North Side (9)	Southwest (10)
1928	7.5	32.4	28.0[c]	....	....	67.9	..	....	....	....
1929	4.3	58.8	58.8	....	....	121.9	413	5,587	13,764[c]	....
1930	3.1	53.8	139.4[c]	....	22.8	219.1	400	4,984	28,504[c]	....
1931	1.5	55.7	196.0	....	85.1	338.3	198	3,984	40,726	....
1932	Plant shut down October 15, 1931	54.7	200.3	....	125.7	380.7	Plant shut down October 15, 1931	4,086	34,658	....
1933		60.8	199.4	....	135.2	395.4		5,788	35,616	....
1934		52.1	193.5	....	128.4	374.0		5,849	39,264	....
1935		37.8[b]	199.4	....	135.2	372.4		4,175[b]	33,673	....
1936		62.7	204.6	....	197.6	464.9		14,387	36,427	....
1937		65.4	204.8	....	275.6	545.8		13,539	37,145	....
1938		61.8	200.1	....	305.7	567.6		11,055	40,387	....
1939		71.6	202.5	158.0[d]	237.8	669.9		10,277	40,288	28,373[d]
1940		67.2	188.2	313.0	361.5	929.9		11,178	37,658	70,020[e]
1941		65.5	208.3	343.0	472.5	1,089.3		10,785	37,361	61,909

[a] Annual average percentages. [b] The old Calumet Plant was shut down from May 9 to September 13, Calumet plant. [c] The North Side plant was placed in operation October 3, 1928, and the North Branch The remaining months of the year were averaged to extend the total to 12 months. In a six-month period B.O.D. record for Calumet is for the settling in the old Imhoff tanks. After 1935 the record is for activated

2. Calumet Works.—This is an activated sludge plant, with a present average capacity of 136 mgd and a maximum of 200 mgd. Sewage was pumped first on December 3, 1935. Treatment with activated sludge began on December 16, 1935. The excess sludge is conditioned, de-watered, dried for burning, or sold for fertilizer.

3. West Side Works.—This is an Imhoff tank plant, designed for an average flow of 472 mgd and a maximum of 700 mgd. The sludge is pumped to open-air drying beds.

4. Southwest Side Works.—This is an activated sludge plant, nominally designed for an average flow of 400 mgd and a maximum of 600 mgd. The first battery of aeration tanks went into service on June 27, 1939. Sludge disposal apparatus has been in operation since August 28, 1939. On March 20, 1940, the Racine Avenue Pumping Station went into service, thus completing the pickup of sewage except for some minor sewers with a flow of about 70 mgd. By September 26, 1940, all but 35 mgd was connected.

The equivalent population of the Sanitary District is as follows (exclusive of the Corn Products Refining Company):

Population	1930	1940	1941
Human (U. S. Census)	3,901,569	3,962,514	3,962,514
Industrial equivalent	1,486,000	2,237,000	2,549,300
Total	5,387,569	6,199,514	6,511,814

The industrial equivalent population for 1930 was estimated. That for 1940 is the actual value for the period from April to December, inclusive. The peak month in 1940 was November, with a population equivalent of 3,065,000.

Expenditures.—To December 31, 1941, about $166,300,000 had been spent by the Sanitary District for the intercepting sewers, pumping stations, and

THE MAJOR WORKS, THE SANITARY DISTRICT OF CHICAGO

IN TONS PER YEAR			(c) REMOVAL OF 5-DAY B.O.D.[a]				(d) REMOVAL OF SUSPENDED SOLIDS[a]				
West Side (11)	Grand total (12)	To channel (13)	North Side (14)	Calumet[b] (15)	West Side (16)	Southwest (17)	North Side[c] (18)	Calumet[b] (19)	West Side (20)	Southwest (21)	Year
....	....	....	....	....	....	....	....	....	....	....	1928
....	19,764	13,359	....	....	....	....	....	....	....	....	1929
2,805	36,693	25,072	92.0	....	....	....	89.1	....	....	....	1930
9,881	54,789	29,092	92.6	....	56.9	....	91.0	....	58.4	....	1931
15,115	53,859	28,203	91.7	31.3	42.0	....	91.3	56.8	51.2	....	1932
19,655	61,059	32,349	88.3	33.4	42.0	....	89.2	55.8	57.5	....	1933
15,796	60,909	32,764	89.4	38.7	47.5	....	90.5	54.7	61.4	....	1934
11,321	49,169	27,395	91.0	42.9[f]	54.5	....	90.2	54.0[f]	61.1	....	1935
18,993	69,807	36,644	92.2	83.5[f]	49.2	....	89.8	86.9[f]	52.9	....	1936
19,274	69,958	42,495	91.5	86.3	40.7	....	91.5	90.1	40.2	....	1937
29,976	81,418	44,198	92.2	83.1	47.4	....	93.0	90.2	54.5	....	1938
29,985	109,133	53,970	93.5	87.0	53.7	85.8[g]	92.4	90.6	63.0	88.5[g]	1939
33,077	151,933	20,224	94.6	90.7	50.0	67.3	92.4	90.1	58.0	73.4	1940
36,019	146,073	0	94.9	90.5	44.3	56.1	93.8	90.8	54.5	64.9	1941

in 1928. It suspended operations September 15, 1935, and beginning with 1936, records are for the new pumping station began operating June 28, 1930. [d] The Southwest plant began pumping May 23, 1939. 28 mgd were pumped. [e] Complete treatment on approximately 60% of the total flow. [f] Prior to 1936 the sludge. [g] Six months.

treatment works. It was estimated that $11,756,900 was required to complete the program. The principal items to be built were at the West-Southwest works—namely, the aeration and final settling tanks and completion of the blower house and sludge handling plant.

PROGRESS IN TREATMENT

Table 3(*a*) shows that the flow of sewage treated increased from 67.9 mgd in 1928 to 929.9 mgd in 1940. Of the latter, 123 mgd was reported untreated in 1940. The total sludge and grit discharged to the Main Channel aggregated

TABLE 4.—DRY VOLATILE SOLIDS DISCHARGED INTO THE CANAL
(Tons per Year)

Solids	1936	1937	1938	1939	1940
Sludge	21,432	24,613	22,450	33,865	14,937
Raw sewage	111,960	90,450	81,490	63,150	20,290
West Side effluent	12,180	20,070	19,160	14,969	28,305
Southwest effluent	0	0	0	3,674	12,940
Total	145,572	135,133	123,100	115,658	76,472

53,970 tons in 1939 (see Table 3(*b*)). In 1938 it was 44,198. On and after September 16, 1940, no sludge and grit was so discharged, because of the provision of some temporary lagoons in addition to the permanent sludge handling

equipment. The efficiencies of the various plants are shown in Tables 3(*c*) and 3(*d*). The 1941 data are from actual records and were not before the Master.

The volatile solids entering the Main Channel decreased from 145,572 in 1936 to 76,472 in 1940 (see Table 4). Briefly for 1938, 1939, and 1940, the percentage reduction in B.O.D. was as shown in Table 5. The Southwest works lacks blowers and sludge handling equipment. Activated sludge treatment was given only to sufficient sewage to produce solids within the capacity of the dewatering and drying equipment. The remainder received preliminary settling treatment only.

TABLE 5.—Percentage Reduction in B.O.D.

Plant	1938	1939	1940	1941
North Side	92.2	93.5	94.6	94.9
Calumet	83.1	87.0	90.7	90.5
West Side[a]	47.4	53.7	50.0	44.3
Southwest	....	....	67.3	56.1
Weighted average			67.8	60.6

[a] This is only an Imhoff tank plant.

Periods of Flow.—For a clearer understanding of the problem in the Main Channel and Brandon Pool, the time factors of flow must be visualized. The actual periods of flow (in hours) from Damen Avenue to Brandon Road Dam, with the average velocities in the Main Channel, are shown in Table 6.

TABLE 6.—Flow Data, Main Channel and Brandon Pool

Month (1939)	Flow (cu ft per sec)	Damen Ave. to Summit		Summit to Lockport		Between Brandon Road Dam and:	
		Hours of flow	Miles per hr	Hours of flow	Miles per hr	Lockport (hr)	Damen Ave. (hr)
Jan.	2,910	30.4	0.26	62.5	0.35	19.0	112.5
June	4,227	24.0	0.33	44.0	0.54	13.0	81.0
June	7,891	11.0	0.72	22.0	1.01	7.0	40.0
June	8,311	11.0	0.72	20.5	1.08	7.0	38.5
....	2,200	62.4	0.13	78.8	0.29	24.8	166.0

Probable Conditions at Lockport and Joliet in the Summers of 1941 and 1942

At the request of the Master, the experts on both sides submitted estimates on the probable conditions at Lockport and Joliet in the summers of 1941 and 1942. A principal factor was the oxygen demand of sludge deposits in the Main Channel and Brandon Pool. This was difficult to estimate.

The difference between the two sides was enlarged by an over-optimistic viewpoint of the Opposing States as to the date of completion of the remaining Sanitary District works. This viewpoint went so far as to assume that equipment could be delivered long prior to the dates set under contract and the works completed in 1941. As a matter of fact, under the present conditions (November, 1942) of National Defense, it is clear that the works cannot be completed in 1942, and unless priorities are granted, not until after the war. At least two years will then be required, after construction is actively resumed.

The Master concluded that the conditions in Brandon Pool would be better in 1941 and 1942 than in 1939. He noted that in the summer of 1938 there was only 0.2 ppm of dissolved oxygen, and that no special complaints were heard. However, he believed that there would be no dissolved oxygen in the pool in 1941 and that offensive odors were likely to recur.

Available Remedial Measures

The contention of Illinois was that the only immediate, feasible, remedial or ameliorating measure available for relief of the critical situation at Brandon Pool is an increased diversion of water from Lake Michigan.

The writer testified that a diversion of 5,000 cu ft per sec would improve the conditions over those existing prior to 1939, and that no temporary measures are available which can be applied quickly to obviate the conditions in Brandon Pool other than an increased diversion. Messrs. Calvert and Mohlman concurred. Messrs. A. M. Buswell and Enslow were confident that chlorination would provide more certain relief at Brandon Pool than the diversion of 3,500 cu ft per sec additional; yet Mr. Buswell admitted that dilution reduces the B.O.D. more certainly than chlorine.

Clarence R. Andrew, Assoc. M. Am. Soc. C. E., stated that an additional diversion would reduce the odors. The only two feasible methods known to him were the removal of the pollution at its source or the addition of sufficient water to prevent the odors. Dr. Carlson noted that an increased diversion would lessen both the odors and the menace to health.

As the witnesses for both sides agreed that the critical period of nuisance and health hazard occurs during the warmer months (from early April through October), and as no complaint or health hazard arises with the 1,500 cu ft per sec diversion in the colder months from November through March, the budgeting of the diversion appeared logical on a graduated scale, with a maximum in midsummer and tapering off into spring and fall, with a minimum in the winter.

Mr. Enslow stated the critical temperature of flow to be 62° F. As the temperature rises above this, bacterial action is stimulated, thus increasing gasification from decomposing organic matter. The temperature record of the water at Lockport, in degrees Fahrenheit, is as follows:

Month	1939	1940	Month	1939	1940
January	41°	35°	July	76°	74°
February	38°	41°	August	77°	73°
March	45°	42°	September	75°	72°
April	56°	50°	October	64°	65°
May	64°	58°	November	53°	52°
June	71°	70°	December	46°	39°

Mr Ramey stated that since December 31, 1938, the Sanitary District has budgeted the water diverted from Lake Michigan in a manner to best meet the needs of sanitation, with a diversion limited to an annual average of 1,500 cu ft per sec. This includes the rainfall runoff from the Chicago and Calumet rivers, amounting to 550 cu ft per sec. There remains a dependable dry-weather

diversion of approximately 1,000 cu ft per sec. In 1939, from May to October, the dry-weather diversion averaged 1,090 cu ft per sec.

For the protection of the water supply of Chicago, Mr. Ramey did not consider it safe to maintain a flow of less than 2,400 cu ft per sec in the Main Channel at Lockport. Domestic pumpage amounts to 1,650 cu ft per sec and the storm-water runoff equals 550 cu ft per sec, making a total of 2,100 cu ft per sec. This leaves only 300 cu ft per sec to come from Lake Michigan.

To provide greater flow during warmer months, dry-weather flows from 2,400 to 2,600 cu ft per sec were budgeted and attained in the cooler months of 1939 and 1940. Operating results can be improved but little with the present limit on the diversion.

Various remedial or ameliorating measures were proposed by the Opposing States. Illinois contended that these were unsound in principle, effective only to a negligible degree, and available too late to serve the emergency in 1941 and 1942. Illinois further believed that the practicability of the suggestions should be tested from the four standpoints of (*a*) proven reliability and soundness, (*b*) degree of relief effected, (*c*) the time required for installation, and (*d*) the cost.

The suggestions may be divided into temporary and permanent classes, as indicated in Table 7.

TABLE 7.—SCHEDULE OF REMEDIAL OR AMELIORATING MEASURES

No.	Program	Opposing States (temporary)	Illinois (permanent)
1	Chemical treatment at West Side works	✓	....
2	Activated sludge treatment for West Side Imhoff tank effluent	....	✓
3	Eliminate by-passing of Southwest preliminary settled effluent	✓	✓
4	Keep all sludge out of Main Channel	✓	✓
5	Chlorinate the flow in Main Channel	✓	....
6	Chlorinate the effluent of the West Side works	✓	....
7	Double the air used at the North Side works; increase the aeration period from 5 hr to 7 hr to produce 5 ppm of nitrogen as nitrate	✓	....
8	Build cascades at the controlling works at Lockport to aerate the flow above Brandon Pool	✓	....
9	Increase the diversion in summer, in accordance with the results of analyses, to provide a minimum of 1.0 ppm of dissolved oxygen in Brandon Pool	....	✓ (temporary)

The Opposing States urged that the first step to improve the Waterway would be to discontinue the discharge of sludge into it. This was actually done at the end of September, 1940. However, considerable putrescible material is carried over in partly treated effluents. Other suggested remedial measures follow.

Dredging, Draining, or Flushing Brandon Road Pool.—The pool is about 4.84 miles long, with a maximum width of 1,400 ft. Its surface area totals about 12,409,500 sq ft. The water depth varies from 10 to 20 ft. The original volume was approximately 151,000,000 cu ft, which was reduced by 1939 to 120,000,000 cu ft. Mr. Andrew of the U. S. Engineer office estimated about 1,250,000 to 1,500,000 cu yd of deposited material, from 2 to 10 ft deep. He stated that increasing the flow would not remove this deposit, but hydraulic dredging could do so, at a cost of between $500,000 and $700,000, in a period of from 6 to 9

months. Louis R. Howson, M. Am. Soc. C. E., thought the cost might be $400,000 and that any nuisance in lagooning the material could be controlled by chlorine.

Experts on both sides agreed that the denser material has largely lost its potency to cause nuisance, and that hydraulic dredging would remove chiefly material no longer the cause of offense. Mr. Howson doubted the practicability of such dredging as compared to chlorination and other methods. Mr. Mohlman and the writer regarded dredging as an impractical proposition.

Draining the Pool.—Draining the pool was regarded as impracticable by the U. S. Engineer office for a variety of reasons. The Chief of Engineers declined to allow it. From the discussion finally emerged a plan for permitting a flow of 10,000 cu ft per sec in addition to domestic pumpage for 10 days beginning December 2, 1940. To this both sides and the Court agreed.

Experimental 10-Day Flushing.—During the 10-day test beginning December 2, 1940, the total flow at Lockport averaged 9,973 cu ft per sec. The net diversion was 8,430 cu ft per sec. The level of the Brandon Pool was lowered 0.5 ft at first and later an additional foot. The flushing scoured out the Main Channel and deposited in the Brandon Pool more solids than were discharged (see Table 8). However, the solids washed out of the Pool were slightly higher in percentage organic matter than those which entered, as the incoming total solids contained 32.4% volatile matter and the outgoing solids contained 35.6 per cent.

TABLE 8.—Results of Experimental 10-Day Flushing of Brandon Pool

Description	Total Suspended Matter		Volatile Suspended Matter	
	Tons	%	Tons	%
Delivered to pool	84,010	....	27,067	....
Discharged from pool	21,123	25.2	7,512	27.7
Remainder deposited in pool	62,887	74.8	19,555	72.3

During the test the velocities in the pool averaged from 0.47 to 0.55 ft per sec. The maximum velocities were from 1.33 to 1.58 ft per sec, and the minimum near the banks from 0 to 0.07 ft per sec. Mr. Mohlman estimated about 524,000 cu yd of material were swept in. All the witnesses agreed that in the Main Channel and Brandon Pool a considerable benefit was accomplished, as a whole, by removing sludge from the Main Channel. Mr. Mohlman doubted if this effect would offset the effect of the additional sludge deposited in the pool.

Chlorination of Waterway.—The Opposing States urged the use of chlorine to eliminate odors at Lockport and Joliet.

Mr. Enslow considered the sludge at Brandon Pool to create a bad situation, which would require from three to four times the chlorine necessary if the sludge deposits were removed. In his opinion there is no parallel case in existence for the use of chlorine to prevent septicity for such long periods of flow in a deep river underlaid by sludge, and chlorination at present would be a failure at

Brandon Pool. However, he suggested chlorination at the Southwest plant and at Lockport. Assuming that no new sludge came into the pool his opinion was that 3 ppm of chlorine was required at the Southwest works and 6 ppm at Lockport, although 3 ppm might suffice. If sludge continued to enter, 12 ppm or more, even 15 or 25 ppm, might be required. He considered the attempt speculative, and estimated the cost roughly at $700 to $3,000 per day, based upon chlorine at $43.20 a ton. The equipment might cost $115,000.

Mr. Buswell deprecated the effect of the sludge deposits. He believed 2 to 3 ppm of chlorine at Lockport would be sufficient, with a chloro-boat in the pool to apply 1 ppm if sludge were no longer discharged, and if sludge discharge was continued maybe 5 ppm would be needed. To obtain an authoritative opinion, he admitted, would require tests for at least three months. Mr. Enslow stated that Mr. Buswell's estimate of 1 ppm was too optimistic, but perhaps 3 ppm would be safe, if applied at two points.

Joseph W. Ellms, M. Am. Soc. C. E., estimated that the cost of a 150-ton chlorination plant would not exceed $300,000 and would require six or seven months to build. He felt that chlorine should be applied before septic conditions developed, and at several points. He stated that chlorine could reduce, and perhaps eliminate, odors from hydrogen sulfide, but that hydrogen sulfide would persist as long as organic matter is fed into the pool.

L. F. Warrick testified that chlorine would have a decided beneficial effect upon the Waterway. As a rough estimate he thought the effect of chlorine applied at the Southwest plant might last from 30 min to 2 hr or longer. (The time of flow in June, 1939, actually varied between 24 hr and 76 hr from the Southwest works to Lockport.)

Mr. Howson testified generally that chlorination "is a very effective and positive method of controlling odors." He considered it practicable and effective to relieve the conditions at Joliet and Lockport, and estimated that 6 ppm of chlorine would suffice. With a total flow of 3,130 cu ft per sec, this would amount to 50 tons of chlorine per day. His testimony on the subject of chlorination was based chiefly on Mr. Enslow's testimony.

The Illinois experts conceded that, in many situations, chlorine is used effectively to prevent odors; but in view of the sludge deposits in the Main Channel and Brandon Pool, they believed that chlorine could not be used effectively to eliminate the odor conditions, even if large quantities were used at great expense.

Mr. Mohlman indicated that at least three points of application would be needed—Damen Avenue, Summit, and Lockport. This might require about 150 tons per day or about $6,000 to $7,000 per day during the summer months. The writer called attention to the fact that chlorine does not dispose of the organic matter that causes the nuisance at Joliet.

Cecil Kirk Calvert, Affiilate Am. Soc. C. E., presented a disappointing experience in 1930, at Indianapolis, Ind., in which an average of 10 ppm of chlorine was applied to the plant effluent. The reduction in B.O.D. was about 10% or 2 ppm of B.O.D. per part per million of chlorine. Usually the chlorine disappeared as soon as the effluent mixed with the river water. On only one occasion was any residual chlorine found farther away than 2,000 ft. He be-

lieved that the estimate of 150 tons per day for the Chicago-Joliet situation was very moderate and the application would be required for five or six months. He did not consider that the expenditure was justified, because the nuisance would not be measurably reduced.

Thus the amounts of chlorine required per day were variously estimated between 50 and 150 tons, with very doubtful assurance that the money would be well spent even as an emergency measure. Witnesses on both sides agreed that there is no water or sewage plant in the United States with a capacity for applying chlorine greater than the Detroit (Mich.) Sewage Works with 27 tons per day, and even this plant has not actually applied as much as 10 tons per day.

Cascades at Lockport.—The flow of the Main Channel passes through the turbines at the power house below Lockport and generates electric power which is worth about $1,500 per day to the Sanitary District.

Mr. Howson proposed that this flow be run over a cascade instead of generating power, thereby picking up 6.75 ppm of dissolved oxygen or about 113,000 lb of dissolved oxygen, equivalent to that contained in 2,650 cu ft per sec of lake water. He estimated the cost at between $50,000 and $100,000 for a wooden structure and a period of three to six months for construction. He based the pickup on the experience at the Hastings Dam in the Mississippi River and in the North Side effluent at Chicago.

Mr. Warrick believed that such cascades would have a beneficial effect, inasmuch as at Hastings Dam, in 1933 and 1934, about 7.8 ppm of dissolved oxygen was obtained. He stated that the B.O.D. in the water at the Hastings Dam was only 7 ppm as compared with 30 or 40 ppm in the flow at Lockport. He declined to express any opinion as to how long such pickup could be retained and doubted whether any dissolved oxygen would be carried through.

Mr. Calvert testified that such cascades would improve conditions but slightly, and that they would create a local odor nuisance.

Mr. Mohlman and the writer conceded that some dissolved oxygen would be picked up, not to exceed 3 ppm, but that the effect would be lost before the flow reached the pool because of its anaerobic condition. (As a matter of fact, actual tests made in October, 1924, showed a pickup over the old Bear Trap Dam of 4.1 ppm, which practically disappeared in the flow of 5 miles to the upper end of Brandon Pool.) The writer admitted that the pickup of dissolved oxygen at the North Side works averaged about 6.7 ppm from 1934 to 1939, but he doubted if all the dissolved oxygen so entrained was effective.

Mr. Howson contended that such dissolved oxygen would not be lost because of the 15-ft fall in the flow from the cascade to the pool. Mr. Enslow agreed with Mr. Mohlman that the re-aeration obtained by cascading would not last long under summer conditions and would not justify the cost. If chlorine were added to the cascade, he felt that some benefit would accrue. Otherwise, it would be just a local odor nuisance.

The writer conceded that if the suggestion of cascading had merit, the effect could be accomplished over the existing control dams at the power house with very small expenditure. He did not consider that enough was gained to justify the loss of $1,500 per day to the Sanitary District. The Opposing States

argued that Illinois should be required to try every measure that might possibly ameliorate conditions even at considerable expense, before diverting any additional water from the Lake.

Supply of Additional Oxygen through the Production of Nitrate by Increased Use of Air at North Side and Calumet Works.—Mr. Howson suggested that 40,500 lb of additional oxygen might be developed in the effluents of the North Side and Calumet works by using more air in the activated sludge process, at an operating cost of $89,000 for a 5-month period, or $216,000 for a 12-month period. Messrs. Warrick, Ellms, and Buswell agreed in general. Mr. Buswell stated that he knew of no streams with nitrites and nitrates present in the flow where septic conditions occurred, as nitrates tended to oxidize hydrogen sulfide and prevent its formation. Under existing conditions Mr. Buswell admitted there would be odors even if nitrates were present.

Messrs. Calvert and Mohlman agreed with the writer that such nitrate production would be of little ultimate value. The production of nitrates would require an increase in detention period in the aeration tanks from the present basis of 4.5 or 5 to 7 hr. Such capacity does not exist at the North Side works. Mr. Mohlman indicated that algae and green plants would form a secondary growth in the North Shore Channel which would create a secondary source of B.O.D. He testified that if an attempt were made to control the odor situation by purchasing sodium nitrate, the cost might reach $17,000 per day.

Chemical Treatment at West Side Works.—Messrs. Buswell and Howson proposed the installation of chemical treatment on the effluent of the Imhoff tanks at the West Side works. This was intended to supplant the treatment by activated sludge which is a part of the Sanitary District permanent program and approved by Messrs. Ellms, Enslow, Howson, and Warrick. Such chemical treatment would reduce the B.O.D. by 21 ppm or, on a flow of 437 mgd, a reduction of 76,000 lb of B.O.D. Mr. Howson estimated that the construction would cost $1,300,000, with an operating expense of from $200,000 for five months to $481,700 for twelve months. (This was substantially the Sanitary District estimate of $1,300,500 (6).) Mr. Howson claimed in September, 1940, that this could be completed by the summer of 1941. According to the Sanitary District report of April, 1940 (6), two and one-half years would be required. Mr. Mohlman and the writer both stated that chemical treatment would not be satisfactory, since it would reduce the B.O.D. at the West Side from 57 to 36 ppm, whereas activated sludge treatment should reduce it to 10 ppm. The Sanitary District estimated that the chemical plant would cost $555,800 per year to operate at the West Side, whereas the activated sludge plant for equivalent purification would only cost $320,600 per year, with a first cost of $3,662,000.

Inasmuch as the Sanitary District has adopted the activated sludge treatment, the Master saw no reason to accept the suggestion that chemical treatment be adopted as a temporary or emergency measure to care for the summer of 1942, at the cost involved, when it would be almost immediately superseded by the superior activated sludge plant.

Chlorination of West Side Imhoff Tank Effluents.—Mr. Howson also described the application of chlorine to the West Side effluent as an alternative

to chemical treatment, as suggested by Mr. Enslow. He estimated that this would reduce the B.O.D. by 50,000 lb instead of 76,000 lb by chemical treatment. The cost of the equipment was estimated at $100,000 and the time for construction at six to nine months. The operating expense was estimated at $105,000 for a period of 150 days. Mr. Mohlman and the writer did not consider chlorination of the West Side effluent a practical measure to ameliorate conditions in the Brandon Pool.

The Master concluded that it was obvious that the suggested work could not be done before 1942 and that the extent of amelioration was limited.

Planned Use of Water.—The decree of April 21, 1930, permits a variation in the diversion, which the Sanitary District has utilized to endeavor to provide more flow in the summer. Mr. Howson suggested that the present arrangement be adjusted to provide 4,200 cu ft per sec flow at Lockport during four summer months.

Illinois conceded the merit of this suggestion but indicated that it was limited by the necessity of providing for storm runoffs to prevent reversal in the Calumet River and flooding of basements in the Chicago Loop area. Hence the Sanitary District carries a reserve for use in the event of storm.

In its revised petition, Illinois proposed to take only 1,150 cu ft per sec of water for January, February, March, November, and December. Assuming that this is adequate, the aggregate difference between 1,500 and 1,150 cu ft per sec for five months is 1,750 cu ft per sec. If distributed over four summer months an additional flow of approximately 437 cu ft per sec is provided for each month, or 1,935 cu ft per sec. If this is added to the domestic pumpage of 1,700 cu ft per sec, the total is 3,635 cu ft per sec, as compared with 4,200 cu ft per sec suggested by Mr. Howson.

The Master indicated that this suggestion would only contribute about 22,000 lb of oxygen to meet the B.O.D. situation at Lockport. The lesser amount possible upon the admission of the Sanitary District is not substantially ameliorating. To quote: "In view of the paramount importance to the health of Chicago of avoiding any reversals of the river" the Master hesitated "to make a finding that the Sanitary District should be required to go further in budgeting than in its modified petition it admits to be possible."

Metering of Chicago Domestic Water Supply.—The Opposing States offered in evidence a table showing that the Chicago per capita consumption was 214% more than the average of nineteen other American cities. In Chicago, of 412,228 water services in 1940, only 115,025 were metered.

Illinois offered evidence through Loran D. Gayton, M. Am. Soc. C. E., then city engineer of Chicago, showing that to provide universal metering would require 300,000 meters, costing in excess of $10,000,000 and requiring more than six years to install, at the rate of 50,000 meters per year. In the 10-yr period from 1930 through 1939, the city installed an average of 4,165 meters per year. The greatest number ever placed in one year in Chicago was 16,864 in 1931. No money had been appropriated for their purchase. Under the circumstances, Mr. Gayton stated that he did not believe that any meters installed in 1941 and 1942 on the South Side area (in connection with the new filtration plant) would appreciably reduce the use of water therein.

Should the domestic pumpage be reduced, the Master found no evidence to enable him to appraise the extent of the resulting benefit.

Activated Sludge Treatment at Southwest Works for West Side Imhoff Effluent.—Mr. Howson suggested that a large part of the Sanitary District program for the activated sludge treatment at the Southwest works of the West Side Imhoff effluent could be speeded up, covering an estimated cost of $3,267,200. The writer testified that this sum was inadequate without the addition of certain essential items, totaling approximately $700,000, thus raising the total to $4,048,000. The Sanitary District estimate for the entire extension of the activated sludge plant was $4,878,500. Furthermore, of the program advocated by Mr. Howson, certain conduits and appurtenances costing about $600,000 would be of no value when the plant was completed.

The Comments of the Master

The Master indicated that the contention of the Opposing States is that the Sanitary District might complete its program before the summer of 1942 by proper effort. He commented that

> "The record indicates that the Sanitary District has been influenced, not so much by the desire to make speed, as by the purpose ultimately to complete an efficient system at as little expense as possible to its taxpayers. These are praiseworthy motives when considered from the standpoint of the Sanitary District alone, but they may not place the emphasis upon expedition to which the legitimate protection of the interests of the opposing States entitles them."

The Master also noted that "the record contains numerous references by representatives of the District to the importance of saving the taxpayers' money as a controlling consideration."

Illinois tried to impress on the Master that with the funds available to the Sanitary District for construction and operation, in accordance with its legal limitations, the engineers of the Sanitary District had endeavored to spend such funds to best advantage to produce permanent results, instead of frittering the money away on expensive makeshifts and thereby injuring the operation of the existing works and delaying the construction of the program which the experts on both sides agreed was the only real solution of the problem, the treatment to a high degree of all the sewage and the handling of all the solids so removed, so that a minimum of residual pollution in solution and in settling solids entered the Illinois Waterway.

Recommendations of the Master

On the basis of his conclusions (see Appendix) the Master recommended that a decree be entered dismissing the petition and the modified petition of the State of Illinois for a modification of the decree of April 21, 1930, and taxing the costs of the litigation against the State of Illinois. The Court then dismissed the petition without comment on May 26, 1941.

Acknowledgment

The writer wishes to acknowledge herewith the free use of the report of the Special Master as well as the record and exhibits in the case, from which liberal quotations have been made.

APPENDIX

Conclusions of the Special Master

After hearing all the testimony and inspecting the upper Illinois Waterway from Brandon Road to the Power House, the Master presented the following conclusions to the Court:

"(1) The actual condition of the Illinois Waterway by reason of the introduction of untreated sewage creates in the summer months a nuisance through offensive odors at Joliet and Lockport, but does not present a menace to health. No nuisance conditions were proven to exist along the Waterway at any other points.

"(2) With respect to remedial or ameliorating measures available to the State of Illinois without an increase in the diversion of water from Lake Michigan, my findings are as follows:

"(*a*) The dredging of Brandon Road Pool would remove chiefly old accumulations of sludge which have completely or largely lost their potency as causes of nuisance and would therefore be of extremely doubtful efficacy. It would cost between $400,000 and $750,000, plus the cost of providing spoil banks and lagoons. It would present problems as to possible nuisance from such spoil banks and lagoons and require further expense for chlorination. I do not think this is a feasible ameliorating measure.

"(*b*) The draining of Brandon Road Pool cannot be accomplished without some interference with navigation, to which the War Department, which has sole jurisdiction over navigation problems, will not consent. For this reason I do not think this suggestion feasible. There would also be presented problems with respect to the water intake pipes of several industries which take water for industrial purposes from the Pool.

"(*c*) Chlorine is an effective measure to reduce and eliminate odors, but owing to the size of the Brandon Road Pool and the large sludge deposits therein and the continuing discharge into the Waterway of incompletely treated sewage, it is impossible to make a reasonably certain estimate of the amount of chlorine which would have to be applied to produce a substantial result.

"In order to have a reasonable prospect of substantially controlling offensive odors, it would be necessary to spend from $3,000 to $4,000 a day for chlorine, plus several hundred thousand dollars for chlorinating equipment.

"(*d*) Cascading the water at Lockport or sending it over the dam there would be remedial to the extent of producing some oxygen at Lockport. How much oxygen would be produced and how much of it would reach Joliet and the Brandon Road Pool is uncertain and could only be determined by actual trial. The use of the water in this way would cost the Sanitary District $1,500 a day in the loss of power and it would be necessary to use an undetermined amount of chlorine to prevent an odor at the point of cascading.

"(*e*) The supply of additional oxygen through production of nitrates by increase of air on the North Side and Calumet plants is not a feasible ameliorating suggestion for the summer of 1941. The evidence before me is not sufficient to prove that it is feasible for 1942, in view of the testimony of the Sanitary District experts that it would require an increase in aeration tank capacity. The proof before me is insufficient to support a conclusion that this suggestion, if it could be put into effect for the summer of 1942, would be substantially ameliorating.

"(*f*) Chemical treatment at the West Side plant would involve a very large permanent expenditure, which could not be made effective in 1941

and if it could be installed by 1942 would be almost immediately superseded by the activated sludge treatment which is provided for by the District's permanent program. This does not seem to me a feasible ameliorating measure.

"(*g*) The estimates as to cost of chlorinating West Side Imhoff tank effluents are too uncertain and the opinions of the experts too conflicting as to the extent of amelioration which it would afford, to enable me to make any finding that such chlorination is a feasible ameliorating measure.

"(*h*) It is feasible for the Sanitary District to budget the 1,500 c.f.s. of water now permitted to be diverted in addition to domestic pumpage so as to divert only 1,150 c.f.s. in the months of January, February, March, November and December, and to allocate the aggregate saving in diversion in those months to the summer months. This ameliorating measure will not, however, materially reduce the BOD at Lockport, and will not, therefore, substantially relieve the odor nuisance.

"(*i*) The adoption of compulsory water metering by Chicago is an ameliorating measure, but the evidence before me is not sufficient to enable me to make any finding as to the extent of amelioration which it would afford or the time within which it could be made available.

"(*j*) The provision of activated sludge treatment at the Southwest plant for the West Side Imhoff tank effluent is a very important and feasible ameliorating measure to which the Sanitary District is committed, and toward which it has made some progress. The extent of the progress will depend upon the industry and enterprise of the Sanitary District. There is no prospect that this ameliorating measure will be operative in the summer of 1941. It is possible, but doubtful, that by special diligence it might be made operative in the summer of 1942 instead of only by the end of 1942, as claimed by the District.

"(*k*) As additional equipment is installed, it will become possible to give complete treatment during the course of 1941 and 1942 to increased quantities of sewage at the Southwest plant. The progress which will be made during these years and the gradually declining influence of sludge deposits from past years will be operative in the summer of 1941 to improve conditions at Lockport and Joliet as compared with 1939 and 1940, and will be operative to a further extent in 1942. The extent of relief from offensive odors which will be afforded at Lockport and Joliet in the summer months of 1941 is very doubtful, but there is a better outlook for the summer months of 1942. Weather conditions will have an important influence.

"Notwithstanding every effort on my part to press the hearings and consideration of this case, one of the three years for which Illinois originally petitioned for relief (1940) has already passed. The people in Joliet and Lockport have submitted to the conditions which prevailed in the summers of 1939 and 1940 without serious consequences to health. If relief is denied for the years 1941 and 1942, the authorities of the Sanitary District may be spurred to increased efforts in the treatment of Chicago sewage. The hearings before me have already resulted in the putting into effect of provisions as to lagooning of incompletely treated sewage which had not previously been adopted."

Bibliography

(1) **"Stabilization of Sewage Sludge Banks,"** by W. Rudolfs, *Indiana Engineering Chemistry*, Vol. 30, 1938, pp. 337–340; abstracted in *Sewage Works Journal*, Vol. 10, 1938, pp. 636–640.

(2) **"Taschenbuch der Stadtentwasserung,"** by Karl Imhoff, Eighth Ed., 1939, pp. 229–231.

(3) **Report of the Committee on Sewage Disposal,** by H. P. Eddy, et al., Engineering Board of Review, The Sanitary District of Chicago, 1925.

(4) **"Chlorination in Sewage Treatment,"** by L. Pearse, et al., Rept. of the Committee on Sewage Disposal, Eng. Section, Am. Public Health Assn., 1933.

(5) **"Preventive Medicine and Hygiene,"** by M. J. Rosenau, Sixth Ed., 1935, pp. 898–900, 928.

(6) **"Report on Additions to Southwest Sewage Treatment Works and Supplemental Treatment for West Side Treatment Works,"** by F. W. Mohlman, L. M. Johnson, W. A. Dundas, L. C. Whittemore, L. Pearse, and H. P. Ramey, made to Chief Engineer, April 16, 1940.

(7) **"Engineering Features of the Illinois Waterway,"** by Walter M. Smith, *Transactions*, Am. Soc. C. E., Vol. 98 (1933), p. 309.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

PHYSICAL PROPERTIES THAT AFFECT THE BEHAVIOR OF STRUCTURAL MEMBERS

BY WILBUR M. WILSON,[1] M. AM. SOC. C. E.

SYNOPSIS

The physical properties of structural steels usually prescribed by specifications are the strength and the ductility. It is reasonable to infer that, if certain physical properties are prescribed, there must be some quite definite relation between these properties and the behavior of structural members. The purpose of this paper is to discuss this relation and to raise the question as to whether or not there are other characteristics of the material that should be considered.

WHY DUCTILITY IS NECESSARY

The two properties universally prescribed (and the only ones usually prescribed) for structural steels are strength and ductility. The value of high strength is self-evident. The value of ductility is so general that some discussion would seem to be in order. Engineers generally agree that ductility is necessary for the following reasons:

(1) To prevent fabrication and erection operations from breaking or weakening the members;

(2) To permit members that are slightly bent in handling to be straightened cold without injury;

(3) To enable plastic deformation to effect a more nearly uniform distribution in regions of concentrated stress due to inaccurate fabrication, thermal stresses due to welding, or other causes;

(4) To reduce greatly the extent to which moderate deformation stresses and load stresses are additive beyond the yield point; and

(5) To give the structure a shock-absorbing capacity that will enable it to withstand accidental shocks of considerable magnitude that are not quantitatively provided for in the design.

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1943.**

[1] Research Prof., Structural Eng., Civ. Eng. Dept., Univ. of Illinois, Urbana, Ill.

All of these actions involve one or more occurrences in limited areas of deformation stress, exceeding slightly the yield point of the material.

(The term "deformation stress" is not in general use and an explanation of its meaning is in order: As the term is used in item 4, a deformation stress is a stress incident to a deformation, as distinguished from a load stress which is incident to a load. The vertical shear in the outstanding leg of the con-

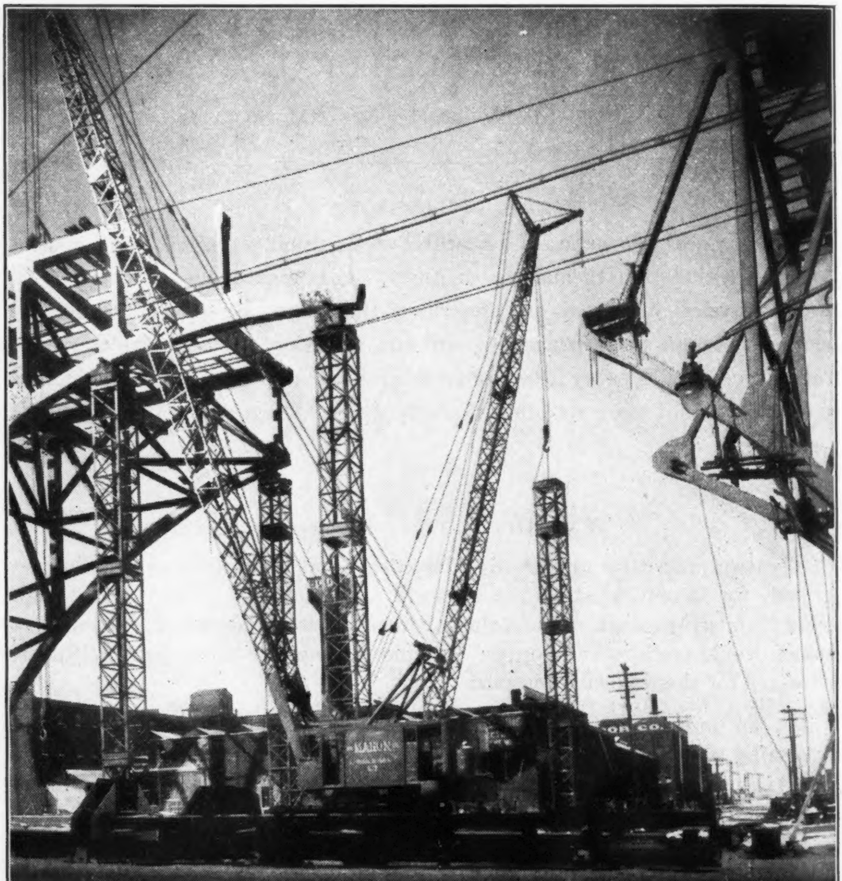


Fig. 1.—Successful Construction Requires the Use of a Steel to Withstand Overstress Without Serious Injury to Members. Construction View of a High-Level Viaduct Over the Cuyahoga River at Cleveland, Ohio

nection angles of a stringer is a load stress because it results from the load carried by the stringer. If the outstanding leg were to fracture because of the shear, the end of the stringer would fall. As distinguished from this action, the outstanding leg of the same angle is subjected to flexure because of the deflection of the stringer. This flexural deformation produces stress; but if

the outstanding leg were equipped with hinges at both vertical edges, the necessary movement could take place without producing flexural stresses in the outstanding leg of the angle, and the connection angle would still hold the stringer in position. This latter case illustrates a primary characteristic of all deformation stresses, which can be stated as follows: If the movement which caused the deformation stress could take place without stress, the load-carrying capacity of the structure of which the member under consideration is a part would not be appreciably affected.)

Hazards of Fabrication and Erection (*Item* (*1*)).—Stresses slightly in excess of the yield point probably occur in most structures during fabrication and erection. Successful construction requires the use of a steel to withstand this overstress without serious injury to the members. For this reason, engineers, in general, consider the capacity of structural steel to elongate 1% or 2% at the yield point, without any appreciable increase in stress, to be one of its most valuable characteristics. This plastic deformation is twenty-five times the elastic deformation corresponding to usual design stresses, but standard A7 carbon steels[2] and a number of low-alloy structural steels can be subjected to this deformation without being stressed appreciably beyond the yield point. Moreover, if a steel originally capable of being elongated 25% is elongated locally 2% during fabrication, it still will retain more ductility than it is feasible to utilize. Furthermore, tests showed that an A7 steel, after being subjected to more than 2,000,000 repetitions of a cycle in which the stress (tension) varied from 25,000 to 50,000 lb per sq in., retained its original static strength, and practically all of its original ductility, although its yield point was raised. (The first application of this stress produced a plastic deformation of the order of 4% or 5%.[3]) There is a possibility, however, that age embrittlement may develop in some steels subsequent to plastic deformation. The possibility of this action taking place in structural steels has not been studied in a comprehensive manner.

For a half century, carbon structural steel similar to A7 has been successfully fabricated by riveting. Low-alloy structural steels have been fabricated, but to a lesser extent. Experience with both of these steels would seem to justify the conclusion that they are suitable for structural purposes after being subjected to the overstress incident to fabrication by riveting. Nevertheless, it would seem desirable, particularly with the low-alloy steels, to make comprehensive studies to determine the extent to which their ductility is reduced by age embrittlement subsequent to plastic deformation.

The foregoing comment pertains to the results of overstress incident to fabrication by riveting. During the past decade the structural engineer has been compelled to accept oxygen cutting and welding as fabrication processes because of their economy and convenience. An A7 carbon steel, with carbon and manganese content limited to 0.25% and 0.70%, respectively, can be welded without serious metallurgical damage, and specifications[4] have been adopted for the welding of highway and railway bridges.

---

[2] Specifications, American Society for Testing Materials, Designation, ASTM—A7.

[3] *Bulletin No. 327*, Univ. of Illinois Eng. Experiment Station, Urbana, Ill., 1941, p. 32.

[4] American Welding Society's 1941 "Specifications for Welded Highway and Railway Bridges."

The effect of oxygen cutting and welding upon structural steels may be summarized as follows:

(*a*) If carbon structural steels similar to A7 are oxygen-cut with a mechanically guided torch, the resulting edges will give tests equal to planed edges, but evidence is not available to justify a similar statement relative to low-alloy steels.[5]

(*b*) Although welding and oxygen cutting, when properly done, do no serious metallurgical damage to carbon structural steels with certain limitations of carbon and manganese content, they may result in serious injury to members subjected to repeated or reversed stresses. Surface irregularities or abrupt changes in section are serious "stress raisers" that greatly reduce the fatigue strength of a structural member. The indiscriminate use of oxygen cutting and welding may result in rough edges or abrupt changes in section, with an accompanying serious decrease in the fatigue strength of the member.

(*c*) Low-alloy steels can be oxygen-cut and welded, provided that the proper procedure is followed, but if the proper procedure is not followed the resulting member may be much more unsafe than a similar member fabricated of carbon steel; furthermore, the possibilities of serious damage are not limited to major operations. The welding of a small, incidental piece to a large member to facilitate erection (a process not supposed to affect the strength of the structure) may embrittle the member or actually cause a fracture before any external load is applied. Similarly, cutting a handhole in the web of a channel or I-beam may embrittle the member. These are the results of metallurgical damage. The damage is all the more dangerous because it results from a minor operation that may not be given serious consideration. No oxygen cutting or welding of any medium-carbon or high-carbon steels or any low-alloy steels should be done except in complete accord with the procedure developed for the particular steel in question, and then only under competent supervision.

Errors in Workmanship (*Item* (*3*)).—The plastic deformation at the yield point that prevents a member from being injured by the fabrication processes also effects a redistribution of uneven stresses and greatly reduces the extent to which load stresses and moderate deformation stresses are additive beyond the yield point. (This might not be true under a tri-axial stress condition.) Errors of workmanship, structural members that must be forced into line, and thermal stresses due to welding are common sources of uneven stress distribution. The mechanism of this stress redistribution is apparent from the behavior of a column whose reinforcement is attached with longitudinal fillet welds. Measurements on columns with an $\frac{L}{r}$-ratio of 70, when reinforced in this manner, indicated that the thermal stresses due to welding were equal to the yield point of the material. Nevertheless, tests showed that the load-carrying capacity of the columns was increased in direct proportion to the increase in section.[6] This was possible because plastic deformation at the yield-point stress prevented the stress at any point from exceeding the yield

[5] *Proceedings*, Am. Soc. C. E., April, 1940, p. 627.

[6] *Bulletin No. 280*, Univ. of Illinois Eng. Experiment Station, Urbana, Ill., 1935, p. 25.

point of the material by an appreciable amount. It also permitted low-stressed material adjacent to high-stressed material to limit the strain of the latter to a value only slightly greater than the elastic strain corresponding to a yield-point stress. Moreover, the high-stressed material was supported laterally by the adjacent low-stressed material. The action of a similarly reinforced tension member is much the same except that the load-carrying capacity of the member might not be increased quite as much as the area of the section because the stress-strain relation would not be the same in the heat-affected zone as in the unaffected zone. However, the difference would not be great for an A7 steel.

The effect of superimposing a load stress upon a deformation stress is similar to the effect of superimposing a load stress upon a thermal stress. In fact, a thermal stress is a deformation stress—a stress resulting from motion, or from the prevention of motion, as distinguished from a stress resulting from a load.

The action of a member reinforced without removing the dead load is similar to the action of a member to which reinforcement is attached with longitudinal fillet welds in that some portions of the section are stressed higher than other portions and in that a redistribution of stress takes place when the maximum stress reaches the yield point.

The stresses in the members used in the foregoing illustration exceeded the yield point of the steel over some portions of the section. The writer has taken the position that, in many situations, this high stress does not endanger the safety of the structure, for the following reasons:

(*A*) In each instance, the stress in excess of the usual design stress is due to a deformation, and this deformation is limited in magnitude to a value only slightly exceeding the elastic deformation corresponding to a yield-point stress by material other than the material that is overstressed; and

(*B*) The deformation at the yield point that equalizes local stress concentrations does not seriously injure carbon and some low-alloy structural steels.

Approval is given to the action described, subject to certain limitations, not with the idea of deliberately overstressing material in new construction, but with the idea of possibly retaining old structures in service which have deformation stresses due to foundation movements or which must be reinforced because of an increase in the loads. Even for this purpose, combined deformation stresses and load stresses equal to the yield point of the steel cannot be accepted for compression members for which the deformation will cause local wrinkling failure or for which eccentricity of the resultant stress will introduce objectionable flexural stresses in the column as a whole. Likewise, a high, combined, load-and-deformation stress cannot be accepted in a tension member that has a reduced section at a point of maximum stress. Such stress will cause the strain resulting from movement to be concentrated at the reduced section instead of being uniformly distributed over a considerable length, as it would be if the section were uniform.

Accidental Shock (*Item* (*5*)).—A fifth reason why ductility is an essential property of structural steels is that it gives the structure a shock-absorbing

capacity that will enable it to withstand accidental shocks of considerable magnitude that are not quantitatively provided for in the design.

Some further explanation would seem necessary to convey a clear conception of this contemplated action. As ordinarily considered in the design of railway bridges, impact consists of the dynamic increment resulting from unbalanced wheels, "out-of-round" wheels, track irregularities, etc. These produce stresses in the members of a bridge, and a member is so designed that when impact stress is added to dead-load and live-load stresses the total does not exceed 18,000 lb per sq in., a value considerably less than the yield point of the material. In other words, the impact that is known to exist, and which is evaluated in the design, produces an elastic deformation of the material. This impact is to be distinguished from an accidental shock (such as might result from a derailment) that is unknown in amount and may be so great that it will bend or break a member. One occurrence of such a shock would cause failure in a tension member made of a brittle material and would cause permanent deformation in a member made of a ductile material. Failure of the brittle member might cause the collapse of the entire structure, whereas permanent deformation of the ductile member, in most instances, would require nothing more serious than its replacement.

In general, engineers recognize the value of a shock-absorbing capacity in a member that will prevent the collapse of the structure as a whole, due to a large accidental shock, but they are not willing to add material to the structure merely to provide safety against this unpredictable action; nor would this seem to be necessary. The proper solution of the problem would seem to be:

First, design the structure so that all loads and forces that are expected to come upon the structure, and which can be evaluated, will be resisted at stresses well below the yield point of the material (this is designing for strength); and

Second, use a material which, in providing the required strength, will also provide the greatest possible shock-absorbing capacity. Then, even though the magnitude of some possible accidental shock cannot be anticipated, provision will have been made to guard against the destructive effect of the shock with the greatest economy.

These two steps, consciously or unconsciously, enter into the design of all structures. The second step is introduced when the use of a ductile material is required. A part of the second step that is not always recognized is that the shock-absorbing capacity of a member depends as much upon the geometrical characteristics of the member as upon the ductility of the material.

The shock-absorbing capacity of an A7 carbon structural steel is approximately one thousand times as great in the plastic range as in the elastic range. This is true only if the entire volume of the member is equally stressed when failure occurs. The truth of this statement is apparent from the stress-strain diagram for such steels. The work done in a tension failure is represented by the area under the stress-strain diagram. This varies for various specimens but may have a value of the order of 18,000 in-lb per cu in. The work done in stretching a steel to 0.8 of the ultimate equals the area under the stress-strain diagram up to a point representing that stress. Since this point is on the

ascending portion of the curve, the area under the portion of the curve to the left of that point will be relatively small—usually of the order of 10% or 15% of the area under the entire diagram.

An effective net section of 80% by test[7] is as great as is usually realized in fabricating by riveting. When a riveted member fails in tension, only a small part of its total volume—that adjacent to a transverse section through the rivet holes—is stressed to its ultimate capacity and undergoes the full plastic deformation of which the material is capable. This small volume absorbs energy that is equal, approximately, to 18,000 in-lb per cu in. Much the greater portion of the volume of the member is stressed to only 0.80 of its ultimate capacity and undergoes a correspondingly small plastic deformation. The energy that this greater volume has absorbed when the weakest section fails is of the order of 1,800 in-lb per cu in. The average value for the entire member depends upon the rivet pattern, but is not likely to exceed 3,000 to 4,000 in-lb per cu in. This is the basis for the statement that the shock-absorbing capacity of a member depends as much upon the geometrical characteristics of the member as upon the ductility of the material. Mechanical engineers, in their treatment of bolts that fail due to shock, have long recognized the principle involved. Such failures are largely eliminated by turning down the body of the bolt to a diameter less than the diameter at the root of the thread.

The fact that some types of members have a greater percentage of gross volume that is effective for absorbing shock than others is the justification for using a less ductile steel for some types of structural members than for others.

Cables of suspension bridges are fabricated of wires that may have a yield point of 150,000 lb per sq in., an ultimate strength of 220,000 lb per sq in., and an elongation in 10 in. as low as 4%. Because the wire is of uniform section, the entire volume will be stressed equally, and the energy required to break the wire is of the order of 7,000 in-lb per cu in., a value twice as great as the entire shock-absorbing capacity of many riveted tension members fabricated from an A7 steel.

The eyebar is another fabricated member that has a high shock-absorbing capacity because the section is uniform over the entire length except for enlargements at the ends. The cables of the Florianópolis (Brazil) suspension bridge were fabricated of medium-carbon, heat-treated eyebars.[8] The specifications for these eyebars prescribed a yield point and ultimate strength of 75,000 and 105,000 lb per sq in., respectively, and an elongation of 5% in 18 ft. The design stress was 50,000 lb per sq in. The energy required to break one of these bars is of the order of 4,300 in-lb per cu in., a value somewhat greater than the corresponding value for a riveted member of A7 steel.

The foregoing examples would seem to justify the use of high-strength, low-ductility steels for wires and eyebars, inasmuch as the uniformity of section more than offsets the low ductility of the material in their effect upon the shock-absorbing capacity of the member. This being true, it would seem that it

[7] "Tension Tests of Large Riveted Joints," by Raymond E. Davis, Glenn B. Woodruff, and Harmer E. Davis, *Transactions*, Am. Soc. C. E., Vol. 105 (1940), p. 1243.

[8] "The Eye-Bar Cable Suspension Bridge at Florianopolis, Brazil," by D. B. Steinman and William G. Grove, *ibid.*, Vol. 92 (1928), p. 280.

might be profitable to consider whether or not it is possible to devise other methods of fabricating structural members so as to conserve a greater part of shock-absorbing capacity inherent in the material instead of discarding from 50% to 80% of this capacity, as is now usually done when fabricating by riveting. Any expedient that is used to stress the member uniformly high over the main part of its length must result in a member that has no section weaker than the body of the member, which is to be of constant section.

Effect of Speed of Loading upon Tensile Properties of Steel

Engineers have formed their conception of the tensile properties of steels from static tests. The term "shock" implies motion; that is, rapidity of application of the force. Some engineers have a vague feeling that the resulting stress, because of its suddenness of application, is more injurious than a static stress of the same intensity. Whether or not there is any basis in fact for this feeling depends upon whether or not the speed at which a steel is strained has any significant effect upon either its strength or its ductility. In the previous section of this paper, the comment relative to the shock-absorbing capacity of members is based upon the assumption that the ultimate strength and stress-strain relations are not significantly affected by the speed of loading.

H. C. Mann[9] reports tension tests on four alloy steels loaded at various speeds, to 28.5 ft per sec. His discussion of the tests contains the following paragraph:

> "It has been shown in the data presented that when similar specimens of the same material are tested at normal temperature, under both static and dynamic conditions, the total energy values obtained from each are the same, provided the velocity of impact is within a certain limiting amount, and that when this velocity is exceeded the impact values become considerably reduced. A possible explanation of this phenomena is obtained from a further consideration of the mechanism of the static test."

The tests cited showed that the shock-absorbing capacity of SAE-1035 steel[10] began to fall off slightly at a speed of 25 ft per sec. In considering this velocity in its relation to what might be experienced in a bridge, it should be noted that the significant velocity is that at which strain occurs rather than that at which a train crosses a bridge or at which one body strikes another.

Mr. Mann used short cylindrical specimens of constant diameter contained between heads of larger diameter, and they were subjected to a high-speed tension test. A few high-speed tension tests were made at the University of Illinois on specimens with geometrical characteristics of structural members.[11] The specimens included 3-in. by $\frac{3}{8}$-in. plates without joints; similar plates with open, drilled holes; similar plates connected with butt welds; and others connected with riveted joints. The force was derived from a 2,300-lb steel block which fell 10 ft and impinged on a nut on the lower end of a 2-in. steel rod

[9] "The Relation Between the Tension, Static and Dynamic Tests," by H. C. Mann, *Proceedings*, A.S.T.M., Vol. 35, Pt. II, p. 323.

[10] Specifications, Society of Automotive Engineers, Designation, SAE-1035.

[11] "The Effect of Speed of Loading Upon the Ductility of Structural Steel," by Gordon L. Jeppesen, and "The Effect of Speed of Loading Upon the Ductility of Structural Steel," by Robert Zaborowski, theses presented to the Univ. of Illinois, Urbana, Ill., in 1938 and 1940, respectively, in partial fulfilment of the requirements for the degrees of Master of Science in Civil Engineering.

suspended from the lower end of the specimen. The results indicated that the shock-absorbing capacity was as great under dynamic loading as under static loading.

The results of these tests are highly significant to the designer of bridges and, although the work that has been done is only exploratory, it would seem that the discussion, "Why Ductility Is Needed," based on static tests, applies to high-speed loading, provided that the "limiting speed" in Mr. Mann's tests is not exceeded. It also would seem that an impact stress is no more destructive than an equal static-load stress, provided that the speed at which strain occurs does not exceed the limiting speed used by Mr. Mann. Additional high-speed loading tests are needed at both normal and low atmospheric temperatures.

Fatigue Failure

Structural members are designed for known loads, treated as though they were static, and are fabricated of a ductile steel to enable them to withstand shock. It is now known that members subjected to repeated or reversed loads may fail by progressive fracture (unfortunately designated as a "fatigue failure") and at a relatively low average stress. Moreover, it is known that the fatigue strength of small machined-and-polished specimens is not a dependable criterion of the fatigue strength of a structural member fabricated of the same material, and that the fatigue strength of a structural member does not necessarily increase with the static strength of the steel from which it is fabricated.[3,12]

The ordinary formulas of mechanics of materials have been developed mainly for the design of structural members subjected to static loads, and they almost entirely neglect to consider localized stresses due to nonhomogeneity of material and sudden changes of cross section in structural parts. There are many localized regions in structural parts where the stress is higher, sometimes several times higher, than that computed by the ordinary formulas for strength of materials.

Any cause which produces a localized area of high stress is called a stress raiser. Abrupt changes in section, notches, reentrant angles, grooves, screw threads, surface irregularities, and discontinuities such as cracks, holes, or inclusions are stress raisers. The effect of a round hole in a plate, and of other simple stress raisers, has been determined by mathematical analysis and studied by means of transparent models viewed with polarized light; but, because the effect of a stress raiser upon the fatigue strength of a specimen is a function of the size of the specimen and of the physical properties of the material, as well as of the stress raiser, these studies are of limited value in predicting the fatigue strength of a particular specimen. They are of real value, however, in determining types of geometrical stress raisers that are to be avoided.

The cumulative experiences of half a century indicate that there is little reason to fear fatigue failures in riveted carbon-steel chords of trusses, or other members subjected to relatively few repetitions of the maximum design stress, provided that the members contain no unusually severe stress raisers. There has not been sufficient experience to justify a similar statement relative to

[12] *Bulletin No. 302*, Univ. of Illinois Eng. Experiment Station, Urbana, Ill., 1938.

welded members in carbon steel or relative to either welded or riveted members of the low-alloy steels. Experience has demonstrated that riveted carbon-steel members subjected to a large number of repetitions of the design stress do fail in fatigue. Fatigue failures of similar details made of low-alloy steels may be expected.

A limited number of fatigue tests have been made on steel structural members.[3,12,13,14] More fatigue tests of both welded and riveted connections are needed, and they should be made on specimens as large and as nearly like the connections used in structures as it is possible to test. Tests are also needed to determine the impact-fatigue strength of structural steels, and these, likewise, should be made on specimens with the geometrical characteristics of structural members.

Shear-Slip Relation for Riveted Joints

The extent to which the shear on a riveted joint is resisted by the friction between the plates depends upon the tension in the rivets. Laboratory tests show that carbon-steel rivets with a grip of 3 in. consistently have a tension of 20,000 to 30,000 lb per sq in.[12] The shear required to produce slip between plates clamped together by such rivets is of the order of 20,000 to 25,000 lb per sq in. of rivet section, a value considerably in excess of the usual working stress for carbon-steel rivets. The tension in some low-alloy steel rivets with a 3-in. grip varied considerably with individual rivets and was as low as 5,000 to 10,000 lb per sq in. for some.[15] The shear required to produce slip between the plates was correspondingly erratic and low. In general, the tension in the rivet increased with the grip in both magnitude and uniformity, with a maximum value somewhat less than the yield point of the steel.

A short, properly driven rivet will fill the rivet hole but probably will have a low tension. Unless the end of a long rivet is cooled before driving, or unless it is tapered, it probably will not fill the hole, but will have a high tension. A rivet that either fills the hole or has high tension will be satisfactory in service if the rivet retains its tension.

The limited number of fatigue tests that are available on this subject indicate that the rivets or bolts of a joint will have a much higher fatigue strength if the shear is resisted principally by the friction between the plates than if it is resisted principally by shear on the rivets or bolts.[16]

Summary and Conclusions

This paper contains ideas, relative to the behavior of structural members, which have resulted from many years of experience in testing engineering materials and structural members. The writer has assumed that the reader is familiar with the literature on the subject but, in a few instances, reference has been made to tests that bore directly upon the particular question being studied. The ideas of the writer, discussed in the body of the paper, may be summarized as follows:

---

[13] *Bulletin No. 310*, Univ. of Illinois Eng. Experiment Station, Urbana, Ill., 1939.

[14] *Bulletin No. 317*, *ibid.*

[15] *Bulletin No. 337*, *ibid.*, 1942.

[16] *Bulletin No. 302*, *ibid.*, pp. 58, 63, and 64.

(1) Structural members should be designed for known forces and then checked against fatigue failure. They should be fabricated of a steel whose ductility, considered in conjunction with the geometrical characteristics of the members, will enable them to resist large, unpredictable shocks.

(2) The plastic deformation that occurs at the yield point without an appreciable increase in stress enables the steel to be fabricated and erected without serious injury, permits redistribution of uneven localized stresses, and reduces the extent to which load stresses and moderate deformation stresses are additive beyond the yield point. Although stresses somewhat greater than the yield point may result at some portions of some sections of the members, they will not reduce the load-carrying capacity of the member appreciably unless the member is made of a steel subject to age embrittlement subsequent to plastic deformation, or unless the conditions are such that the high stresses will cause buckling or local wrinkling of a compression member or excessive elongation at reduced sections of tension members. Additional information is needed relative to the susceptibility of structural steels to age embrittlement subsequent to plastic deformation.

(3) The shock-absorbing capacity of a member depends upon the geometrical characteristics of the member as well as upon the ductility of the steel.

(4) Impact stresses are no more destructive than static stresses of the same magnitude unless the rate of strain exceeds the "limiting speed" of the material, except for the possible effect of impact fatigue, about which practically nothing is known. Additional information is needed relative to the "limiting speed" of structural steel at various temperatures and when fabricated into specimens with various geometrical characteristics.

(5) Oxygen cutting and welding may be used in the fabrication of steel structures, but only under the most careful and enlightened supervision.

(6) Fatigue failures are not likely to occur in riveted carbon-steel chords of trusses, or other members subjected to relatively few repetitions of the maximum design stress, provided that the members contain no unusually severe stress raisers. There has not been sufficient experience to justify a similar statement relative to welded members of carbon steel or relative to either welded or riveted members of low-alloy steel. Carbon-steel members subjected to a relatively large number of repetitions of the design stress have failed in fatigue. Fatigue failures of similar details made of low-alloy steels may be expected. Additional fatigue tests of riveted and welded structural members of both carbon and low-alloy structural steels are needed. Impact-fatigue tests are particularly needed.

(7) A short, properly driven rivet will fill the rivet hole but probably will have a low tension. A long rivet, unless its end is cooled before driving, or unless it is tapered, will not fill the hole but will have a high tension. A rivet that either fills the hole or has high tension will be satisfactory in service if the rivet retains its tension.

ACKNOWLEDGMENT

Fig. 1 was supplied through the courtesy of Link-Belt Company.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

ORGANIZING AND FINANCING SEWAGE TREATMENT PROJECTS

By Samuel A. Greeley,[1] M. Am. Soc. C. E.

Synopsis

The creation of sewage treatment projects is limited, in this paper, to organizations established for the single purpose of sewage treatment and having this limited objective. According to administrative functions, organizations are considered under three classes:

1. Those which function as an arm of the state;
2. Those which function as an arm of a municipal corporation; and
3. Those which are created as new independent, overlapping municipal corporations.

According to geographical character, organizations are considered as local, metropolitan, or regional. The cause, creation, and administration of these special organizations are described with special reference to the Illinois Enabling Act of 1917.

Methods of financing sewage disposal projects are discussed with special reference to revenues from service charges and to loans secured by the revenue. The fundamental considerations in the computation of a charge for sewage disposal service relate to the three principal kinds of sewage for which the service is rendered—sanitary or domestic sewage, storm water, and trade waste. Methods of evaluating these services in terms of a rate of charge and some reference to court decisions and commission rulings are included.

General Statement

The subject of this paper divides itself into two interrelated parts—organization and financing. These functions are governmental and political. Therefore, trends in the functions are influenced by motives and ideas other

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1943.**

[1] (Greeley & Hansen), Chicago, Ill.

than, and as well as, those of the engineering nature. However, the application of engineering yardsticks and experiences to the results of the organization of sewage treatment projects is useful and enlightening. A great variety of methods for acquiring and financing sewage treatment projects exist throughout the United States. Sanitary districts under the Illinois Enabling Act of 1917 have been successful generally. There has been a trend toward the use of revenue bonds which is likely to continue.

Meanings and Descriptions

The meaning of the word "organization," as used in this paper, requires some description. Citizens may consider that they are organized for the creation of sewage treatment projects when they have become incorporated into a city, town, or village whose charter provides the necessary procedure. Many sewage treatment projects have been undertaken, financed, and operated by this kind of organization. This paper, however, is limited to special organizations created for the one purpose of sewage treatment and having this limited objective. Such special organizations have been created under enabling acts in many of the United States. They differ by having different administrative powers, functions, and procedures, or by being different in geographical character.

In general, special organizations for sewage treatment projects fall into three classes of administrative functions and procedures. One class comprises organizations created to function as an arm of the state. This class remains under the general control of the state; its administrative officers are appointed by the state; its expenditures are so controlled; and it reports to the state. The Metropolitan District Commission of Massachusetts is an organization of this type. A second class is one created as an arm of the city, county, or other municipal corporation. This class functions under the control of an existing municipal corporation and, in general, its administrative officers are appointed by the mayor, or by the municipal governing bodies who also control the expenditures and to whom it reports. The Milwaukee Sewerage Commission is an organization of this type. A third class is one created as a new, independent, overlapping municipal corporation. This class, once it has been created, functions independently of any other local corporations or governing boards. Its administrative officers are either appointed, or elected, but it controls its own expenditures and reports only to the public. Special organizations of this type are The Sanitary District of Chicago, the Minneapolis-Saint Paul Sanitary District, and the Twenty-six Sanitary Districts in Illinois, which have been organized under the Enabling Act of 1917.

The classification of special organizations, in accordance with their geographical character, includes local, metropolitan, and regional districts. Local organizations are those having the same boundary and area as an existing municipal corporation, such as the Buffalo Sewer Authority, the Gary Sanitary District, and the Sanitary District of Indianapolis.

Metropolitan organizations are those comprising a large city, the adjoining municipalities, and, perhaps, some area of unincorporated land. Among these

are The Sanitary District of Chicago, the Metropolitan Sewerage Commission of Milwaukee County, and the Minneapolis-Saint Paul Sanitary District.

Regional organizations are those comprising a large area, including a number of municipalities, not necessarily contiguous, and often including a considerable amount of county, township, or unincorporated area. There are relatively few regional organizations for sewage treatment projects. Examples are as follows: North Shore Sanitary District, Illinois, Washington Suburban Sanitary Districts, Maryland, Hampton Roads Sewerage Commission, Virginia, and Passaic Valley Sewerage Commission.

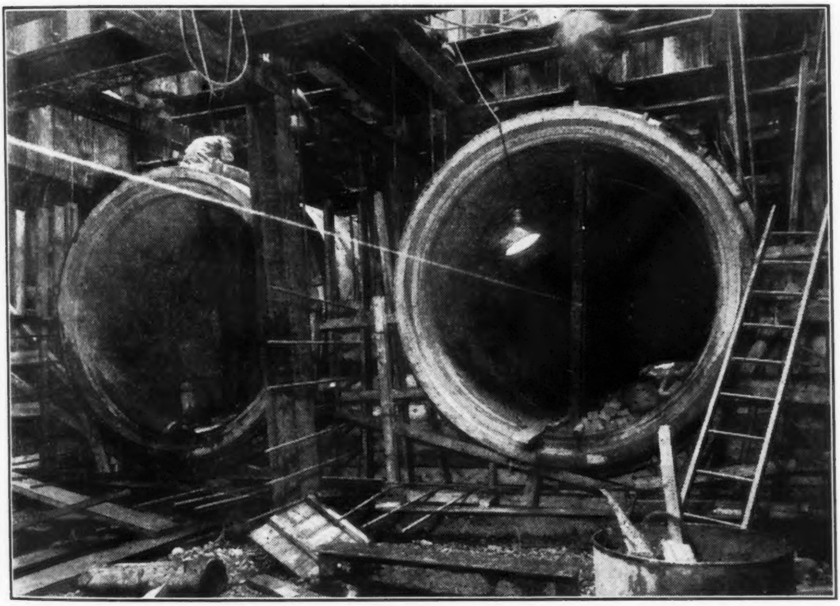


FIG. 1.—METROPOLITAN ORGANIZATIONS ARE THOSE COMPRISING A LARGE CITY, THE ADJOINING MUNICIPALITIES, AND, PERHAPS, SOME AREA OF UNINCORPORATED LAND. THE MINNEAPOLIS-SAINT PAUL (MINN.) SANITARY DISTRICT IS A METROPOLITAN ORGANIZATION OF THIS TYPE. VIEW SHOWS CONSTRUCTION ON AN EXTENSION OF THIS SYSTEM

The foregoing include only the principal kinds of special organizations. In California, for instance, organization is effected through contracts between adjacent municipal corporations. In some cases, special boards or commissions have been created by municipal governments to accomplish a specific sewage treatment project. In 1925, the River Conservancy District Act was passed in Illinois for the special organization of areas co-extensive with the drainage basins of rivers. In general, the trend in the organization of sewage treatment projects has been independent of other local governments.

A major reason for the creation of special organizations has been to secure taxing and financing powers additional to those permitted incorporated municipalities by constitutional restrictions. Thus, under the Illinois Enabling Act of 1917, the sanitary districts have separate powers to issue bonds and

levy taxes under the constitutional limitation of the state, in addition to those which can be levied under the constitutional limit by the component cities, towns, and villages. In states with constitutional limitations, special enabling acts cannot give this additional power direct to an existing municipality. Thus, a strong motive behind the creation of special organizations arises from the fact that they permit the undertaking of difficult, highly specialized, and relatively expensive projects. The Illinois 1917 Act provides administration through the appointment, by the county judge, of three trustees who serve for the nominal salary of $300 per year. This permits the responsibility to be centralized on competent and trusted citizens who have no other duties. Thus, if special legislation is required in any event to provide for financing the project, a public desire to secure a special organization also, more or less outside the routine political field, may be fulfilled.

Special organizations have been called sanitary districts, sewerage commissions, authorities, and joint meetings. The term "sanitary district" is used in this paper. Most sanitary districts have power to finance, construct, and operate sewage disposal projects, including intercepting sewers, pumping stations, sewage treatment plants, and appurtenances.

Status in Illinois

Table 1 shows the dates of the enabling acts and of the creation of early sanitary districts. The first one (Item 1, Table 1) is the Boston Main Drainage,

TABLE 1.—Dates of Early Special Organizations for Sewage Disposal Projects

Item	Special organization	Enabling act	Creation of district
1	Boston (Mass.) Main Drainage	1876	1876
2	Sanitary District of Chicago, Ill.	1889	1889
3	North Metropolitan Sewerage District (Mass.)	1889	1889
4	South Metropolitan Sewerage District (Mass.)	1899	1899
5	Passaic (N. J.) Valley Sewerage Commission	{1902 1907	1902 1910
6	North Shore Sanitary District (Ill.)	1911	1913
7	Decatur, Ill.	1917	1917

which was created by an Act of the Massachusetts General Court in 1876. Acting under authority of the city council, the Mayor of Boston petitioned the Legislature, and obtained the desired authority in an Act to "empower the City of Boston to lay and maintain a main sewer, discharging at Moon Island in Boston Harbor * * *." The second is The Sanitary District of Chicago, Ill., under an enabling act which became law in 1889, the creation of the district taking place the same year. The State of Illinois has had a long and complete experience in the organization of sewage treatment projects. The North Shore Sanitary District (Item 6, Table 1) was organized in 1913, following an Enabling Act of 1911. The first of the Illinois Sanitary Districts under the Enabling Act of 1917 was at Decatur. Thus, since 1889, Chicago and the State of Illinois have had actual experience with the special organization of projects for sewage disposal. Item 5, Table 1, refers to the Passaic (N. J.)

Valley Sewerage Commission. The original enabling act of 1902 was declared unconstitutional and the creation of the district was held in abeyance until another act was passed in 1907. The project was finally consummated in 1910. Table 2 lists eleven states in which sanitary districts have been organized. So far as the writer knows, no other states have organized sanitary districts. It should be noted, in the first place, that the record shows sixty sanitary districts in operation in the United States, with a total population of about fourteen million; 46% of the organized sanitary districts by number, and 36% by population, are in Illinois.

TABLE 2.—RECORD OF SPECIAL ORGANIZATIONS FOR SEWAGE DISPOSAL PROJECTS BY STATES

No.	State	Districts (No.)	Population[a]
1	California	1	260,000
2	Illinois	27	4,764,027
3	Indiana	4	623,512
4	Massachusetts	3	1,911,828
5	Maryland	4	288,125
6	Minnesota	1	805,031
7	New Jersey	8	1,594,366
8	New York	4	1,654,448
9	Ohio	1[b]	....
10	Wisconsin	4	901,693
11	Virginia	1	270,000[c]
12	Total	58	13,073,030

[a] Approximate population served in 1940. [b] Ten county sewer districts, but no sewer works. [c] 1933 population; the 1941 population was estimated at 450,000.

Table 3 is a brief summary of the progress and present status of sewage disposal in a number of states, in the United States as a whole, and in the State of Illinois. Table 3(*a*) shows the number of sewage treatment plants and the approximate population served in all of the eleven states having sewage treatment projects prior to 1900. There were no sewage treatment plants in any of the other states. In these eleven states there were forty-five sewage treatment plants with an approximate population of 546,000. As of 1940 (Table 3(*b*)), the total estimated population in the United States served by sewage treatment projects has increased to about forty million. The population served in the eleven states which had sewage projects prior to 1900 amounts to nearly

TABLE 3.—STATUS OF SEWAGE TREATMENT PRIOR TO 1900 AND IN 1940

No.	State	(*a*) PRIOR TO 1900[a]		(*b*) AS OF 1940[c]	
		Plants (No.)	Population[b]	Plants (No.)	Population[b]
1	Connecticut	3	41,000	54	840,000
2	Illinois	6	50,000	261	5,120,000
3	Iowa	1	1,500	281	795,000
4	Kansas	2	9,300	164	520,000
5	Massachusetts	15	147,000	43	781,000
6	Nebraska	1	7,200	103	280,000
7	New Jersey	7	43,000	156	2,600,000
8	New York	2	8,000+	269	5,400,000
9	Ohio	6	63,000	203	3,045,000
10	Pennsylvania	1	?	181	1,720,000
11	Rhode Island	1	176,000	11	400,000
12	Total	45	546,000	1,726	21,501,000
13	Entire United States	45	546,000		40,000,000

[a] From data compiled by Langdon Pearse, M. Am. Soc. C. E., and his staff, and from other sources. [b] Approximate population served. [c] From U. S. Public Health Service and from other records.

twenty-two million in 1940. Prior to 1900, the population served by sewage treatment plants in Illinois was about 10% of the total so served in the United States, whereas in 1940 it was about 24%.

PART I. ORGANIZATION

Special organizations for sewage treatment projects usually have been created because of the magnitude and difficulty of the project and the need for special financing. Civic leaders have been prompted either to initiate an enabling act, or to lead the community to avail itself of existing legislation. Therefore, sanitary districts are organized to obtain funds beyond the statutory limitations of the municipality or component municipalities, and, in many cases, to provide for a continuing qualified personnel to administer and accomplish the project.

Initiation and Creation

In a general way, sanitary districts are created by enabling acts of the state legislative bodies. Such enabling acts are either mandatory or permissive. Those which are mandatory bring the district into existence with the enabling act, or following the appointment of trustees or commissioners. All of the Illinois Acts are permissive. The area to be organized must petition for an election to determine the question of organizing a district. This step, of course, takes careful account of the rights and prerogatives of the small or local municipality and insures that the state does not usurp any of the local rights.

A recently created sanitation district, known as the Hampton Roads Sanitation Commission, in Virginia, is a regional organization. It was created by an enabling act which provided for an election to determine the question of establishing the Commission. Upon a favorable election, the Commission came into existence when the Governor appointed the five Commissioners provided by the Act. However, the local interests were still further protected by a provision in the enabling act which permits any individual municipality within the sanitary district to declare its intention of withdrawing within six months after the creation of the Commission. Such municipalities as declare their intention to withdraw must, within the next following six-month period, show to the satisfaction of the State Board of Health and the Commission that they have taken the necessary steps to finance, construct, and operate an acceptable sewage disposal project. During these two six-month periods, the Commission cannot take steps to issue bonds. Thus, in addition to the right to vote on the creation of a sanitary district, the local communities are afforded an opportunity to decide whether they will proceed by themselves, or jointly within the sanitary district.

The trend is to continue to safeguard the rights, powers, and interests of the local municipalities.

Area To Be Included

The boundary of the area to be organized is generally included in the originating petition and fixed by the court or other public body hearing and reviewing the petition. In some cases the boundary lines have been fixed by

considerations of expediency, such as municipal corporation limits, property lines, and the like. Some enabling acts require a competent preliminary engineering investigation and report, the purpose of which is to assist the reviewing body in determining the area to be included. There is no such requirement in Illinois, but the trend seems to be to give more consideration to the preliminary aspects of organization. When the boundary of a sanitary district has been fixed and bonds issued and money used for permanent construction, it is difficult to diminish the area of the district. It is easier to annex than to disannex. Amendments have been made to existing Acts, providing for annexation or disannexation.

Administrative Board or Commission

In Illinois, the administrative board is elected in the case of The Sanitary District of Chicago, and is appointed by the county judge in all of the other districts. The principal question is the relative merit of election. The size of the district and of the project is likely to have some bearing, but the trend seems to be toward the appointment of the board. The Hampton Roads Sanitation Commission is composed of five members appointed by the governor. The Buffalo (N. Y.) Sewer Authority, one of the recent large special organizations, comprises five members appointed by the mayor.

Another question of interest relates to the compensation of members of the administrative board. In general, this compensation has been nominal. The Sanitary District of Chicago has nine Trustees, three of whom are elected every two years for a six-year term. This is a very large complex undertaking and requires considerable time by the Trustees. Therefore, they are compensated on a relatively liberal basis. In the other sanitary districts in Illinois, each Trustee now receives $300 per year. The members of the Buffalo Sewer Authority receive no compensation. The Chairman of the Hampton Roads Sanitation Commission, who is designated as a "full time member," may receive not more than $6,000 per year. The other members receive a small compensation for attendance at meetings, not to exceed $300 in any one year. The trend is toward a relatively small compensation.

Methods of Financing

The sanitary districts in Illinois are empowered to levy and collect taxes and to issue bonds upon favorable vote by the people. Amendments to the 1917 Act and others provide that the district may apportion and collect an additional charge for the disposal of industrial sewage and may also levy special assessments upon benefited property for the construction of collecting sewers. None of the districts in Illinois can establish and collect charges for sewage disposal service. The Buffalo Sewer Authority and the Hampton Roads Sanitation Commission are authorized to issue revenue bonds and, in fact, are limited to this method of raising money. The trend is to consider a sewage disposal project as a utility, to be operated on a revenue basis.

The Illinois sanitary districts are empowered to levy and collect taxes as soon as they are organized. Although considerable time may elapse before tax money comes in, it is possible for the board to issue tax anticipation warrants,

or to borrow moderate sums for administrative expenses. In this way, much-needed engineering and legal services can be engaged promptly. The Hampton Roads Sanitation Commission is limited in this regard by the provision that the appropriation made by the state, except in the amount of $10,000, is not available to them before they are empowered, by an election, to issue bonds. The Buffalo Sewer Authority had no means of securing funds until they issued and sold revenue bonds, the first money being advanced by the Public Works Authority (PWA). Fortunately, however, they were able to use facilities provided by the city. Administrative boards should not be so limited in their early available funds as to handicap the progress of completing the project for which they were created.

Powers

The powers granted sanitary districts are not unusual and are more or less similar throughout the United States. The district is given power to engage engineers, lawyers, accountants, and clerks, to build and operate structures, to acquire right-of-way and property, to make contracts, and to formulate and enforce rules and regulations for the operation of the project. A special power, recently granted, is that of receiving and using grants from the federal government. Many enabling acts permit the district to contract with other districts or municipalities for sewage disposal.

Rules and Regulations

Enabling acts, in general, permit sanitary districts to adopt rules and regulations for the use of the facilities which they provide, but do not ordinarily give them police powers. This is understood to be true as regards The Sanitary District of Chicago. The police power rests with the state. The extent to which sanitary districts can regulate the use of their works has not been thoroughly tested, but, in general, the soundness of the rules and regulations has led to their acceptance by the public. The importance of such rules and regulations and their effect is obvious; for instance, it is important that sanitary districts have control over the construction of new sewers which, in turn, will require sewage disposal service.

General Trend

In recent years few sanitary districts have been organized for sewage disposal projects. In Illinois, the most recent important sanitary district was that at Danville in 1934. Since that time, only three small districts have been organized. Throughout the United States, there are relatively few new organizations, the most recent being the Hampton Roads Sanitation Commission, created in 1941, and, among large projects, the Buffalo Sewer Authority created in 1935. Two reasons are (1) the availability of federal grants and (2) the powers given to municipalities to issue revenue bonds. The experience in Illinois in the administration of sanitary districts under the 1917 Act has been so good that this method of procedure could well be extended to other parts of the United States.

Special Considerations

The creation and organization of sanitary districts in the various states and within each state vary in detail. A few illustrations are pertinent, as follows:

(*a*) The Passaic Valley Sewerage Commission is authorized to formulate projects, but cannot begin to create them until a prescribed number of municipalities within the district have approved the undertakings and obligated themselves to furnish the necessary funds.

(*b*) The East Bay Municipal Utilities District, in California, may purchase or construct only revenue-producing utilities. It cannot purchase or construct such utilities until their acquisition has been first approved by a majority of the voters of the district.

(*c*) The Washington (D. C.) Suburban Sanitary Commission may construct and maintain sewage disposal projects, as well as other utilities, and may extend them into adjacent territory outside of the district, wherever the property owners agree to such an extension. It may make and enforce plumbing regulations and build sewer connections directly to houses. It is empowered to borrow money and to levy a tax or to make special assessments and service charges. Appeals from assessments and service charges can be taken to the Maryland Public Service Commission.

An outline of the items included in the Illinois Enabling Act of 1917 is given in the Appendix.

PART II. FINANCING

The following methods of financing have been used by special organizations for sewage disposal projects:

(*a*) Taxes and borrowings through general obligation bonds;
(*b*) Funds contributed by constituent municipalities within the district;
(*c*) Special assessments on benefited property;
(*d*) Appropriations by the state;
(*e*) Voluntary contributions; and
(*f*) Revenues from service charges and loans secured by the revenues.

As the trend in financing has been toward service charges and revenue bonds, the discussion in this part of the paper will be limited to that method.

History of Revenue Bonds

The use of revenue bonds and charges for sewage disposal service is not new, having been resorted to as early as 1890. However, there are relatively few court decisions and rulings by regulatory commissions. From the viewpoint of general theory and the computation of rate structures, the application to sewage disposal projects is relatively undeveloped. There is need for a broad review of experience, and a development and presentation of procedures for review by courts and commissions. Among the early rate structures are those at Brockton, Mass., and Spokane, Wash., which were established between 1890 and 1900. The Brockton project is still financed by the collection of

sewage disposal charges and is a good illustration of a sound rate structure. A survey of the United States made toward the end of 1938 indicated that more than 600 municipalities in thirty-five states were using revenue bonds and the income from sewage disposal charges for financing such projects. There is some confusion in reporting, classifying, and comparing these revenues. In some cases, the rates are sufficient to produce a revenue to meet operating expenses only, and in such cases the debt service is paid by taxes from the general fund. In other cases, the rates are sufficient to produce revenues sufficient for the entire annual cost of the project, including operation and debt service and, in some cases, a reserve or revolving fund for current extensions and replacements. Whenever rates are compared, the items covered by the annual revenue should be stated.

The power of municipalities to establish sewage disposal charges comes from special enabling acts or from sections in acts for establishing special organizations. The various acts grant different powers as to the amount and use of revenue. For instance, the Ohio law includes the following provision:

> The municipality may establish charges and equitable rates or charges of rents to be paid by those whose premises are sewered by a connection to the sewage disposal works, and these charges constitute a lien upon the property sewered by such a connection. The revenue shall be used for the payment of the cost of management, maintenance, operation and repair of the project, and any surplus in such fund may be used for enlarging or replacing parts of the project and for the creation of a sinking fund for the payment of any debt incurred for the construction; it shall not be used for the expansion of the system into unsewered areas.

Other acts provide that the revenues may be used not only for the expenses of operation and maintenance, but also to meet the debt service and to provide a reserve for improvements.

The use of revenue bonds received a considerable impetus from the grants donated to municipalities by PWA. During its active life, PWA aided in the construction of 1,524 sewage disposal projects, of which 230 were financed by revenue bonds. A brief summary of the sums involved follows:

Source of fund	
PWA loans, 22%	$101,500,000
Grants, 37%	171,500,000
Non-federal, 41%	189,000,000
Total	$462,000,000
Loans secured by revenue bonds	
PWA, 26%	$ 26,400,000
Non-federal, 14%	26,500,000
Total	$ 52,900,000

It is of interest to note that about 104 municipalities in Canada are reported as having established sewage disposal charges. It is reported that some 200 municipalities in Texas receive sewage disposal service from projects financed by revenue bonds, and there are a number in Massachusetts and New Jersey.

About 26 municipalities in Illinois have financed sewage disposal projects by revenue bonds and service charges.

Discussion of Terms

The term "sewage disposal project" is used in a broad sense to include all degrees of sewage treatment, all of the structures included in the project, as it may have been financed, and, in some cases, the construction of sewers only. In short, it is a comprehensive term describing projects in the field of sewerage and sewage treatment which have been financed by revenue bonds. In some cases, the charges are referred to as sewer rentals[2] or sewer service charges. In the writer's opinion, the term "sewage disposal charge" is the better term.

Fundamental Considerations

In general, there are three principal kinds of sewage for which service is rendered, as follows:

Sanitary or domestic sewage;
Storm water (usually comprising the first part of the runoff); and
Industrial sewage or trade waste.

The structures to provide the service will include combined sewers or separate sewers, and the service charge may be limited to a sewage disposal project for sanitary or domestic sewage only, or for combined sewage. This is an important fundamental consideration. In some cases, industrial sewage may be excluded, but, in general, much industrial sewage is discharged into the system. Thus, in considering revenues and rates, the kind of a sewage disposal project must be stated. On this basis, the elements of use or service are the following:

(*a*) Treatment and disposal of sanitary or domestic sewage;
(*b*) Handling and disposal of storm water; and
(*c*) Treatment and disposal of industrial sewage.

The collection of these kinds of sewage may or may not be part of the project, but a consideration and comparison of rate schedules should take the element of service and its cost into consideration. In addition to the foregoing, the municipality must make some provision for future growth and future population, and this is an element of service. Assuming that these are the proper elements of use or service, an effort should be made to measure them and to evaluate or price the quantity of the service. A quite general practice is to consider that sanitary or domestic sewage is reasonably measured by the quantity or use of water in the dwelling, exclusive, perhaps, of water used for lawn sprinkling, and discharged to the sewage disposal project.

Industrial sewages may affect the cost of providing sewage disposal service by their relatively greater strength, as compared with domestic sewage. They may have a relatively high oxygen demand or chlorine demand; they may contain large quantities of suspended solids and grease, difficult to handle;

[2] See "Sewer Rental Laws and Procedure," Third Progress Report of the Committee of the Sanitary Eng. Div., *Proceedings*, Am. Soc. C. E., November, 1942, p. 1585.

or they may be discharged in batches, and not uniformly, throughout the 24 hours. Some industrial sewages may contain toxic or corrosive substances. Unless these additional elements of service and their cost are covered in the rate structure, it is possible that a charge of discrimination could be supported on behalf of the domestic user. Fortunately, however, in many municipalities the amount of such annual cost and revenue is not a large proportion of the total, and adjustments can frequently be made by special rates or agreements with industries, in some cases providing pretreatment or equalization structures at the industry.

If the collecting sewers are combined, some (and occasionally much) of the storm water reaches the sewage disposal project. Capacity and treatment for this storm water must be provided and a part of the total annual cost is thus required for storm-water disposal. This service is more a function of the tributary area than of the use of water. Properties of greater area will shed a larger quantity of storm water to the sewers and require a greater sewer capacity, and to this extent a higher cost for disposal. Properties having more impervious surface than others will discharge relatively greater quantities of storm water; and properties of value, where interruptions to their use are costly, require a more extensive and expensive service. Thus, area is indicated as a possible yardstick for measuring this service. The evaluation of this yardstick, in terms of a sewage disposal charge or rate, so far has been computed from the assessed valuation, which gives some weight to the extent, area, and use value of the property. Based on present experience, a charge, on the basis of area and value, appears to be reasonable and expedient.

A fundamental consideration of service is to provide future capacity for the growth and development of the city. Such additional expenditures and annual costs are made for the benefit and use of presently vacant property and for an increased development and use of presently occupied property. Property, therefore, is served by the sewage disposal project. Its use and value are protected and stabilized. This element of service may also be related to area, perhaps through the assessed valuation.

Thus, for combined systems in which the financial structure covers both storm and sanitary sewers, a two-part rate is indicated. One part of the rate might be based on the use of water and the other part on the assessed valuation of property within the area served, or to be served, by the project. Such a two-part rate has been in use at Buffalo for more than three years and has recently been under review by the courts in the case of Philadelphia. Further, to accomplish a reasonable and equitable rate structure, the ordinance should provide for special charges, agreements, or considerations as regards industrial sewages.

Court Decisions

The writer is not aware of many court decisions related specifically to sewage disposal projects. An early one is that of Carson versus Brockton Sewerage Commission, decided by the Supreme Judicial Court of Massachusetts on May 27, 1901. The principal contention of the petitioner was that, having

paid an assessment for the installation of the sewers, he could not be forced to pay a special annual rental charge. The Court decided in favor of the City, on the basis that a sewer rental, computed on the quantity of water or the amount of the water bill, was not unreasonable. The Brockton revenue for sewage disposal service is in part a usage and in part a tax levy. The use charge brings in about 60% of the total annual revenue.

In 1942, there was a decision by the Court of Appeals of Richland County, Ohio, in connection with Gatton versus City of Mansfield. The appellant sought relief from the City for shutting off water furnished to her premises, because of her delinquency in payment of sewer rentals. The Court sustained the City. Recently, the Supreme Court of Pennsylvania has considered the case of a proposed sewer rental ordinance for Philadelphia, which establishes a two-part rate, comprising three mills for each dollar of the fair value of the property served, and an amount equal to one-quarter of the water rent. A very complete argument was prepared by the city solicitor, which included references and quotations from many pertinent court decisions, a few of which relate quite directly to sewage disposal service of one kind or another.

Commission Rulings

There are a number of sewage disposal utilities in the United States which are owned and operated privately, most of them being in New Jersey and in Texas. The New Jersey Board of Public Utility Commissioners has ruled on charges established by some of these private companies, as, for instance, the Lakewood Water Company. The rate approved by the Commissioners comprised a service charge related to the size of the water meter, a consumption charge related to the use of water, and a minimum charge. The service rendered is understood to have been mainly for domestic sewage and not for storm water.

Types of Rate Schedules

There are many types of rate schedules for sewage disposal service,[2] among which are the following:

(*a*) A charge related to the water bill, which may be a percentage of the water bill or may be a separate graduated or uniform scale, related to the quantity of water.

(*b*) A flat rate per connection, according to the class or kind of property.

(*c*) A charge based on the number and kind of fixtures connected to the project.

The kind of rate structure best suited to any community depends upon its size and complexity, whether the services are for sanitary or combined sewage, the number of water meters installed, the amount of revenue to be realized, and what it covers, and other local factors. The trend seems to be toward rates or charges related to the water consumption, with consideration being given to a two-part rate, where this is feasible and appropriate, under the Enabling Act, to local conditions.

Experience with Service Charges

A good indication of the feasibility of financing revenue bonds by sewage disposal charges is the relation between the cash collected and the total amount of the individual bills. Experience in the percentage of collections appears to have been favorable. Table 4(*a*) is taken from the annual reports of Lima, Ohio (population in 1940—44,711).

TABLE 4.—Experience as to the Percentage of Sewer Rentals Billed Actually Collected

Year	Total billed (dollars)	Total Received	
		Dollars	%
(*a*) Lima, Ohio			
1932[a]	27,982.15	26,488.42	94.7
1933	49,054.50	46,244.71	94.3
1934	34,427.09	34,308.29	99.7
1935	33,974.39	33,901.54	99.8
1936	35,287.41	36,581.08	103.7
1937	36,044.23	36,511.83	101.3
1938	35,991.52	36,048.14	100.1
(*b*) Buffalo, N. Y.			
1939[b]	1,505,004	1,410,254	93.8
1940[b]	1,456,449	1,354,586	91.1
1941[b]	1,485,932	1,395,132	94.0

[a] Nine-month period. [b] Year ending June 30.

Experience at Battle Creek, Mich., indicates that current arrears do not, in general, exceed 5% of the bills. In 1936, the bills totaled $52,821, and the arrears $4,459, or 8.4%.

The experience at Buffalo has also been satisfactory. The sewage disposal charges were established to realize a total annual revenue of $1,500,000. The book record of revenue for the three fiscal years of operation is as shown in Table 4(*b*). The actual cash collections have been in excess of 93% of the current bills. Some proportion of the arrears has been in deferred payments by adjacent tributary municipalities, with whom contract adjustments were pending. The general experience with the administration of sewage disposal charges has been satisfactory. There seems also to have been a steady demand for revenue bonds so financed, at reasonable interest rates—generally below 4%.

General Summary

There has been a marked trend toward the use of revenue bonds and charges for financing sewage disposal projects. Experience in selling revenue bonds and collecting revenue has been satisfactory. It seems likely that this method of financing will continue and expand. There is need for a further clarification of the structure of rate schedules for sewage disposal service and a classification of present experience in the light of fundamental considerations.

APPENDIX

Illinois Sanitary District Law, Act of 1917 As Amended 1919, 1923, 1927

The subjects of various sections, and brief comments on "An Act to Create Sanitary Districts and to Provide for sewage disposal," are as follows:

1. (As amended 1927.) Any area of contiguous territory containing one or more incorporated cities, towns, etc., which can construct and maintain treatment plants, may be incorporated as a sanitary district under this Act in the manner following:

(a) Petition to county judge by 100 legal voters to submit to legal voters question of whether territory shall be organized into sanitary district;
(b) County judge will then call two judges of the circuit court, and the three shall constitute a board of commissioners to consider boundaries;
(c) A 20-day notice of a meeting of Commissioners shall be published;
(d) After a meeting and hearing, the Commissioners fix boundaries; and
(e) Finally, the case is decided by an election within 60 days after step 1(d).

2. Courts must then take judicial notice of the existence of the sanitary district.

3. A board of trustees, consisting of three members, for the management, control, and government of the affairs and business of each sanitary district organized under this Act, shall be created in the following manner:

(a) The county judge will appoint trustees, who must be residents of the district. Not more than two shall be appointed from one incorporated town, village, etc., and for terms of 1, 2, and 3 years.
(b) (As added in 1919.) The county judge is given power to fill any vacancy.

4. (As amended in 1927.) The powers of the board are to manage and control all the affairs and the property of the district.

5. Ordinances.

6. Proof of ordinances.

7. (As amended in 1927.)

(a) The Board has power to provide for the collection and disposal of sewage and drainage, to maintain conduits, pipes, pumps, to contract with other sanitary districts, and to treat and purify sewage—"provided, nothing herein contained empowers the board to operate a system of water works for the purpose of furnishing or delivering water to any municipality or the inhabitants thereof." The Board is not authorized to flow sewage into Lake Michigan.
(b) Its duty is to provide sewers and treatment plant; a penalty is provided for violation.
(c) It may collect, from producers of industrial waste, the costs of fair additional construction, maintenance, and operating expense, over and above those covered by normal taxes.

8. It may acquire real or personal property by purchase, condemnation or otherwise, etc.

9. It may borrow, issue bonds, etc., but not to exceed 5% on the valuation of taxable property. The question must be submitted to a vote at an election.

10. It may impose a tax to pay indebtedness.

11. Contracts must be let to lowest responsible bidders.

12. (As amended in 1927.) The Board may levy additional taxes after an election has been held to authorize such levy; the tax can be terminated by a later election. The 1927 amendment provides for the certification and collection of taxes, the deposit of funds, etc.

13. Sewers, channels, etc., may cross public property, with the approval of the governor.

14. The Board may contract for sewage from a U. S. military post within its bounds.

15. Condemnation of private property.

16. Condemnation of property held for public use.

17. Power to permit outside territory to use its drains, etc.

18. (As amended in 1919.) Power to prevent pollution of water supply.

19. (As added in 1923.) Power to construct drains, sewers, pumping stations.

20. (As added in 1923.) Special assessments—annual installments.

21. (As added in 1923.) Bonds to anticipate collection of assessments.

22. (As added in 1923.) Cost of acquiring private property to be included in assessment, according to state local amendment act.

23. (As added in 1927.) Additional contiguous territory may be added to any sanitary district organized under this Act, as follows:

(*a*) Petition by 10% of the legal voters of the addition to submit to legal voters of addition such proposal, taxpayers thereof to assume proportionate share of bonded indebtedness, provided no territory disqualified in Section 1 of this Act is included.

(*b*) County judge to call all the county judges in the sanitary district and proposed addition, which body shall constitute a board of commissioners to have power and authority to consider boundaries.

(*c*) Notice of and hearing as provided in Section 1 of this Act.

(*d*) Trustees of district to accept by ordinance annexing the additional territory.

24. (As added in 1927.) Any contiguous territory within boundaries of district may become disconnected from such district in the following manner: Petition by 10% of legal voters of territory seeking disconnection to submit question to legal voters. County judges form commission. Such territory must have no bonded indebtedness incurred while it was a part of the district which has not been paid up, nor any special assessments. Notices and hearings are practically the same as in Section 1 of this Act.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

SEISMIC SUBSURFACE EXPLORATION ON THE ST. LAWRENCE RIVER PROJECT

By E. R. Shepard,[1] Esq., and Reuben M. Haines,[2] Assoc. M. Am. Soc. C. E.

Synopsis

The seismic method of subsurface exploration was used extensively by the Corps of Engineers, U. S. Army, in preliminary studies of the St. Lawrence River Project in the International Rapids Section. A well-defined interface between the glaciated rock and the overlying till made possible the determination of the depth of overburden with satisfactory accuracy. Considerable success was also attained in determining changes in the overburden. The program included tests in both quiet and swift water, for which special apparatus and methods were devised. The use of the seismic data enabled the design of concrete structures, navigation channels, and hydraulic cuts to proceed more rapidly than if exploration consisted only of drill holes. By correlation with drill hole data and the general geology of the region, the results aided in determining the extent and depth of various deposits of glacial till and other types of overburden and also their condition of density and compaction. The subaqueous tests are believed to be unique for obtaining information of the type required for this project. As the results of the subaqueous exploration were satisfactory for this project, the use of the seismic method should be broadened in future investigations to include similar conditions.

Introduction

Late in 1940 a program of subsurface investigations was inaugurated to obtain data for the design of structures for the St. Lawrence River Project by the Corps of Engineers, U. S. Army, with a district office at Massena, N. Y. The project, extending a distance of approximately 45 miles, is located in what

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1943.**

[1] Senior Physicist, Corps of Engrs., U. S. Army, Washington, D. C.

[2] Engr., Corps of Engrs., U. S. Army, Massena, N. Y.

is known as the International Rapids Section of the St. Lawrence River. Because of the large number of concrete structures, dikes, navigation channels, and hydraulic cuts, the problem of making adequate subsurface explorations assumed major proportions. Prior to 1940, sufficient studies and investigations had been made by the United States and Canadian governments, and various private companies, to determine the general plan of the project and the approximate location of each element. The data, however, were far short of those which would be required for determining the most feasible and economical locations and designs for the various elements and for making cost estimates.

At the outset of the present exploratory program it was recognized that subsurface conditions were favorable to the seismic method of exploration, that the interface between the hard rock and the overburden could be determined with fair accuracy by that method, and that some success might be achieved in determining changes in the overburden. From the previous subsurface and geological surveys, it was known that the bedrock in the region is of sedimentary origin, consisting mainly of dolomite and limestone. Glacial action had removed the residual soft, weathered rock, leaving a generally smooth, undulating, hard rock surface of low relief, broken by occasional valleys, ridges, and faults. The overburden consists of deposits of variable glacial till which appears in the form of elongated ridges at the surface and deposits of clays, silts, and uniform sands in the valleys between the till ridges.

The purpose of the seismic program was twofold: (1) To guide the drilling program and thereby limit it to obtaining necessary and essential data; and (2) to augment the drilling program with a large number of less expensive tests in areas where the required information could be obtained by that method. Extensive tests were made at sites where very little prior information was available, particularly in the numerous areas where cuts, channels, and canals were projected. Tests were also made to obtain data between drill holes. Considerable work was done in the river channel as well as on land. At the Long Sault Dam site, seismic tests were made in very swift water where drilling would have been exceptionally expensive and hazardous.

Except during March and April, when the ground was frozen, the explorations were conducted almost continuously from November, 1940, to October, 1941. During this time 408 lines were explored on land, 38 in quiet water, and 11 single-shot determinations were made in very swift water. Of this number 300 lines were explored to obtain information for the layout and estimates of canals, cuts, and channels. The seismic field party consisted of an instrument operator, one authorized dynamite man, and six helpers. The field crew completed from 2 to 6 lines per day according to weather conditions and the accessibility of the locations. Film records were developed daily by a local commercial photographer.

Theory and Application of Seismic Method of Exploration

Principles Involved.—A comprehensive discussion of the principles involved in seismic exploration will not be given, as this subject is treated adequately in a voluminous literature on the subject. A brief description of the theory is

necessary, however, if the significance of the data presented herein is to be understood.

The seismic method of exploration is based on the fact that the velocity of wave propagation in the earth's crust differs greatly in different substances. Granular and plastic materials, such as sand, clay, and gravel, transmit wave disturbances at velocities of roughly 800 to 8,000 ft per sec, whereas rigid rock transmits such disturbances at 10,000 to 20,000 ft per sec. This wide range in velocities in different kinds of soils and rocks is largely dependent on their elastic properties, a factor closely related to rigidity and hardness. To what extent other physical properties, as moisture content, texture, compaction, void space, cementation, and homogeneity, may be determined from velocity measurements is not fully known, although they do enter into the problem.

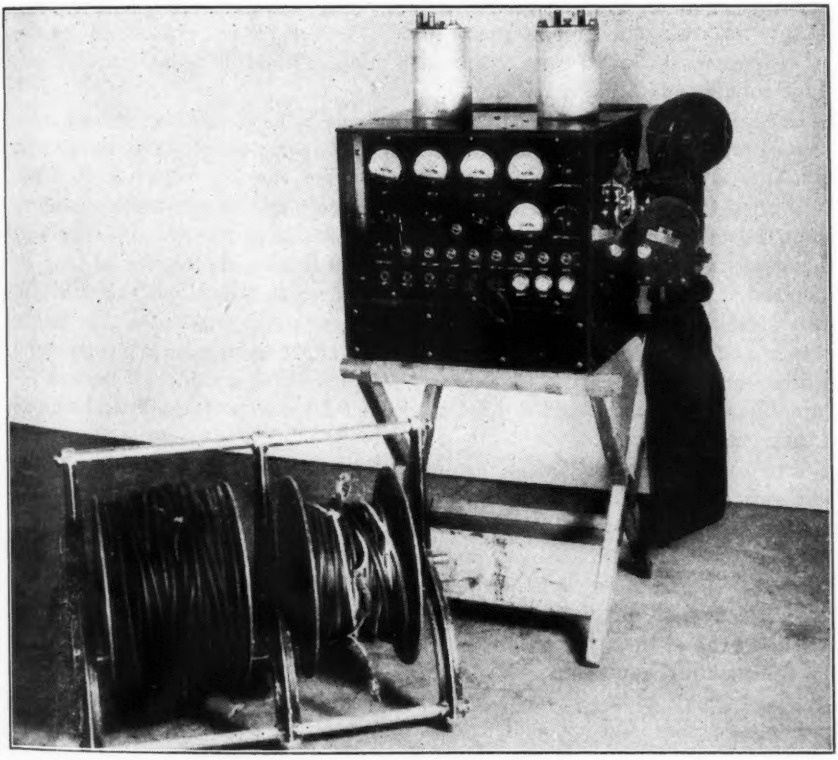


FIG. 1.—PORTABLE SEISMOGRAPH

Apparatus.—The seismic apparatus used for subsurface exploration on engineering projects consists essentially of one or more detectors or geophones and an oscillograph recorder. The detectors are connected electrically with the oscillograph so that the time of arrival of a wave disturbance at each detector

may be recorded photographically. The disturbance is created by exploding a charge of dynamite in the ground at a known distance from the detectors. An electrical connection between the oscillograph and the detonating circuit records the shot instant. A time scale, made by an electrically driven tuning

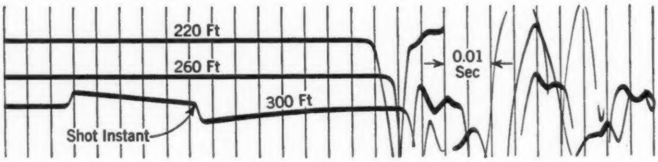


FIG. 2.—TYPICAL THREE-CHANNEL SEISMIC RECORD

fork, is recorded on the oscillograph film. The seismic apparatus used in this investigation, shown in Fig. 1, was of a simple, portable type developed by the Public Roads Administration.

Application of Method.—With the type of apparatus used on the St. Lawrence River Project the usual field procedure is to place three detectors on the ground in a line, at intervals of 40 ft. Dynamite charges varying from 0.25 to 2.5 lb are buried to a depth of 3 ft and fired successively and at increasing distances along the detector line, beginning at 10 ft from the center detector and extending the shooting distance by intervals of about 50 ft to such lengths as may be required. A maximum shooting distance of three to four times the depth to which information is desired is usually necessary. Where possible the seismic lines are laid out along a surface contour in an effort to stay on one type and a nearly uniform depth of overburden. Although these conditions are not required for interpretation of the data, more accurate interpretations can be made if they exist.

From the film records, a sample of which is shown in Fig. 2, the time of the first arrival of wave travel from each shot to each of the three detectors is read and the results plotted in the form of a time-distance graph. In a homogeneous material the time-distance relation will be a straight line through the origin. This is evident from the fact that in such a medium the velocity of wave propagation is constant, or the time of travel is proportional to the shooting distance. It is also evident that the reciprocal of the slope of such a graph is a measure of the velocity in the medium.

When a layer of homogeneous soil is underlain by one through which waves travel at a higher velocity, such as that designated as clay in Fig. 3, there will be a critical distance, OF, for which the time of travel through the upper medium is just equal to the time of travel over the longer path which penetrates the lower medium. At this critical distance there will be a break in the time-distance graph to a different slope, CD, which represents the velocity of wave propagation in the second medium. In like manner, when the critical shooting distance, OG, is passed, the first arrivals will be through the high-velocity rock and the time-distance graph will assume the new trend, DW, the slope of which

determines the velocity in the rock stratum. From the ordinates OA and AB on the time axis and the velocities indicated by the slopes of the several elements of the graph, it is possible to calculate the thickness of each stratum of overburden.

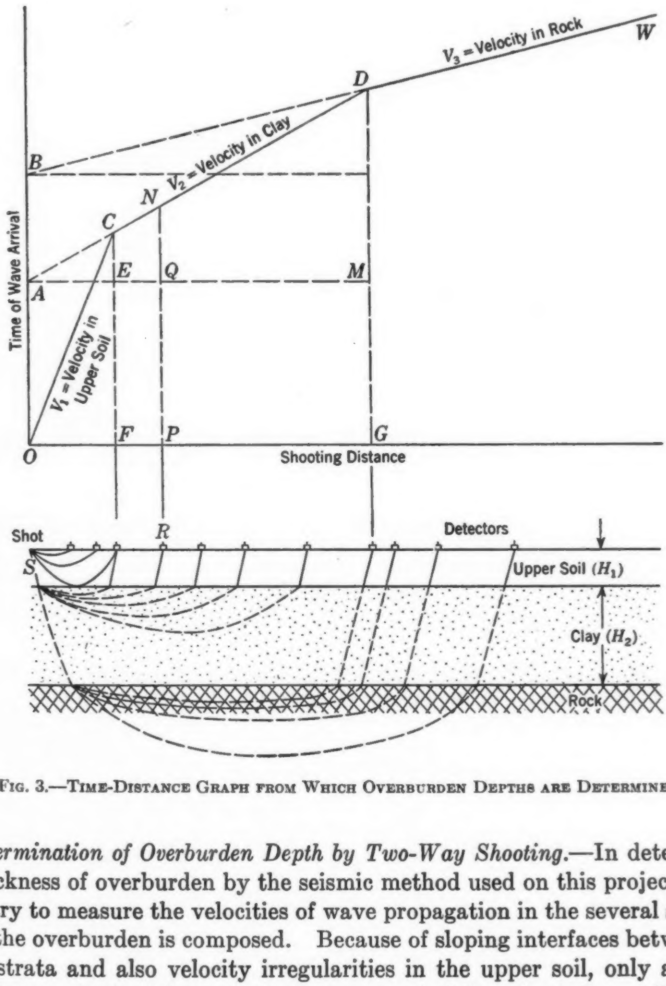


FIG. 3.—TIME-DISTANCE GRAPH FROM WHICH OVERBURDEN DEPTHS ARE DETERMINED

Determination of Overburden Depth by Two-Way Shooting.—In determining the thickness of overburden by the seismic method used on this project, it was necessary to measure the velocities of wave propagation in the several strata of which the overburden is composed. Because of sloping interfaces between different strata and also velocity irregularities in the upper soil, only apparent velocities are obtained from shots along one end of the seismic line. To obtain true velocities, therefore, it was necessary to shoot against the detector setup from opposite directions. The true velocities in the deeper strata were then found by taking the harmonic mean of the respective apparent velocities. When shooting "up-dip" along a sloping interface the apparent velocity in the deeper medium is higher than the true velocity, and when shooting "down-dip" the apparent velocity is less than the true velocity. Variations in moisture

content, compaction, and other physical properties of the soil immediately beneath the detectors give rise to similar effects which might easily be interpreted as indicating a sloping interface. Where erratic and divergent data are obtained as a result of such conditions, it is only by a careful analysis of all data pertaining to a given area that errors of interpretation can be avoided. Considerable experience, good judgment, a knowledge of overburden conditions as they exist in the field, and a thorough understanding of the principles involved are required if satisfactory results are to be obtained by the seismic method of exploration.

After the velocities have been determined the actual computations of the depths of the several strata are relatively simple. The formulas by which depths are computed are given by Maurice Ewing, A. P. Crary, and H. M. Rutherford.[3] These formulas in the forms most frequently used are as follows:

$$H_1 = \frac{T_1 V_1}{2 \cos \alpha} \qquad (1a)$$

$$H_2 = \frac{T_2 V_2}{2 \cos \beta_2} - \frac{H_1 V_2 \cos \beta_1}{V_1 \cos \beta_2} \qquad (1b)$$

and

$$H_3 = \frac{T_3 V_3}{2 \cos \gamma_3} - \frac{H_1 V_3 \cos \gamma_1}{V_1 \cos \gamma_3} - \frac{H_2 V_3 \cos \gamma_2}{V_2 \cos \gamma_3} \qquad (1c)$$

in which (see Fig. 3): H_1, H_2, and H_3 = thicknesses of various layers; V_1, V_2, and V_3 = wave velocities in corresponding layers; T_1, T_2, and T_3 = the time intercepts on the Y-axis, OA, OB, etc.; α, β, and γ = angles of refraction; $\sin \alpha = \frac{V_1}{V_2}$, $\sin \beta_1 = \frac{V_1}{V_3}$, $\sin \beta_2 = \frac{V_2}{V_3}$, $\sin \gamma_1 = \frac{V_1}{V_4}$, $\sin \gamma_2 = \frac{V_2}{V_4}$, and $\sin \gamma_3 = \frac{V_3}{V_4}$.

In some instances, particularly for two or three layer formations, it is possible to use a simpler form of computation which gives values closely approximating those derived by the standard method.[4] In Fig. 3 the shorter formulas become:

$$H_1 = \frac{\overline{\mathrm{OA}} \times V_1}{2 \cos \alpha} \qquad (2a)$$

$$H_2 = \frac{\overline{\mathrm{AB}} \times V_2}{2 \cos \beta_2} \qquad (2b)$$

RESULTS OF EXPLORATIONS ON LAND

Typical Time-Distance Graph.—The exploration program on the St. Lawrence River Project included seismic determinations in the river channels as well as on land. The work on land was done in accordance with the aforementioned

[3] "Geophysical Studies in the Atlantic Coastal Plain," *Bulletin of the Geophysical Society of America*, Vol. 48, June 1, 1937, pp. 753–802; see also "The Seismic Method of Exploration Applied to Construction Projects," by E. R. Shepard, *The Military Engineer*, September–October, 1939.

[4] "Application of the Seismic Refraction Method of Subsurface Exploration to Flood Control Projects," by E. R. Shepard and A. E. Wood, *Technical Publication No. 1219*, Am. Inst. of Mining and Metallurgical Engrs., June, 1940.

methods of field procedure and analysis of data. A typical time-distance graph, derived from data obtained on the right abutment of the Long Sault Dam site, is shown in Fig. 4 and illustrates the method of plotting the data. With three detectors spaced at 40-ft intervals along the shot line, three points on the graph were obtained from each shot, as shown in Fig. 4. Shots ahead of

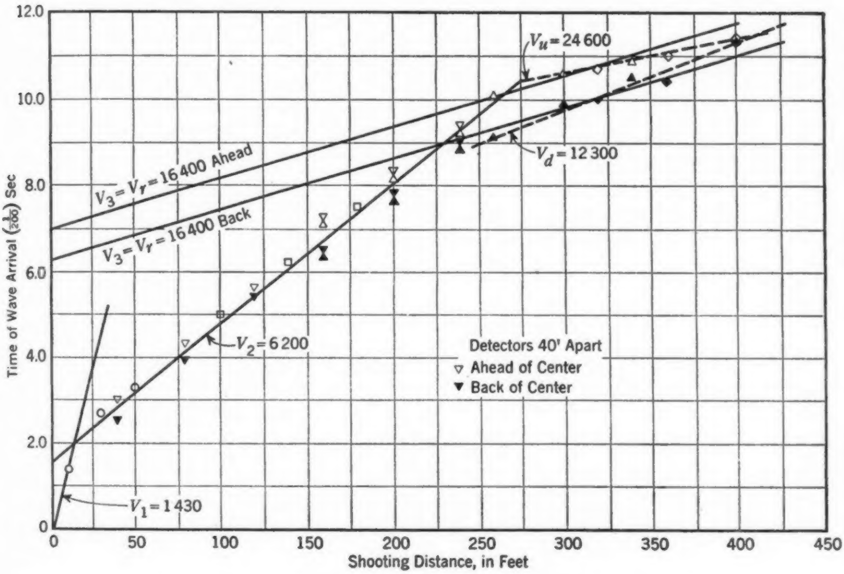


FIG. 4.—TYPICAL TIME DISTANCE GRAPH (DETECTORS 40 FT APART)

the center are shown by open characters, and shots back of the center are shown by corresponding closed characters. From a study of the graph it will be seen that for shooting distances as great as 15 ft, first arrivals were through the upper soil, in which the velocity was 1,430 ft per sec. For shooting distances between 15 and 250 ft first arrivals were through a compact material in which the velocity was found to be 6,200 ft per sec. For shooting distances greater than 250 ft, the arrival times were shorter by way of the deep rock than through the overburden. The apparent velocity in the rock, V_u, when shooting from the "ahead" end of the line, or up-dip, is just twice as great as the apparent velocity, V_d, when shooting from the "back" end of the line, or down-dip. The true velocity in the rock, V_r, was determined by taking the harmonic mean of the apparent velocities and was found to be 16,400 ft per sec.

The depth of overburden at the center of the line was computed as follows, using the velocities previously determined and the approximate method for which Eqs. 2 are employed. The overburden depths are computed as follows:

$V_r = \frac{2 \times 24{,}600 \times 12{,}300}{24{,}600 + 12{,}300} = 16{,}400$; $\cos \alpha = \frac{V_1}{V_2} = 0.97$; and $\cos \beta_2 = \frac{V_2}{V_R} = 0.925$. Then:

Computation	Feet
By Eq. 2*a*, $H_1 = \dfrac{1.6 \times 1{,}430}{400 \times 0.97} =$	5.9
One half the shot depth $= \dfrac{3}{2} =$	1.5
Depth of upper soil =	7.4
By Eq. 2*b*, the depth of till, H_2 (ahead) $= \dfrac{5.34 \times 6{,}200}{400 \times 0.925} =$	89.5
Depth to rock (ahead) =	96.9
Similarly, since H_2 (back) = 77.5, the depth to rock (back) $= H_{1+2}$ (back) =	84.9
The average depth to rock at the center $= \dfrac{96.9 + 84.9}{2} =$	90.9
Elevation, top of ground =	238.1
Elevation, top of rock =	147.2

From the computations it will be seen that the depth of the upper-soil layer, H_1, in which the velocity of wave propagation was 1,430 ft per sec, is 7.4 ft; T_1 in this instance is the interval on the time axis between the origin and its point of intersection with the V_2 limb of the graph, or 1.6; H_2 (ahead) or the thickness of the glacial till under the "ahead" end of the line was found to be 89.5 ft; and T_2 in this case is the difference between the intercepts of V_3 (ahead) and V_2 on the time axis, or 5.34. Similarly, H_2 for the back half of the line was found to be 77.5 ft. The total depths of overburden ahead and back are 96.9 and 84.9 ft, respectively. The depth at the center is assumed to be the average of these two values, or 90.9 ft. Theoretically any depth is determined by taking one half the sum of the depths at the shot point and the detector or receiving point, neither of which depth is known. The recorded depth for any line was the average of the mean or normal depths so determined for the two ends of the line. Because of the prevalent variation between the apparent and true velocities in most of the areas investigated, it was believed unwise to attempt to determine slopes in the rock surface from apparent velocities or to predict multiple rock elevations along a shooting line, and only one rock elevation on each line has been recorded in the final analysis of the data. The recorded elevation was designated as that at the center of the line.

Determination of Overburden Conditions.—In most locations considerable success was attained in classifying the overburden with respect to hardness and compaction as well as in determining the elevation of the top of rock. In areas where the characteristics of the overburden were generally known, the seismic data were useful in estimating the depths of individual strata. The seismic data have been used in studies of overburden conditions at structure sites and where excavations will be made for proposed hydraulic cuts and navigation channels.

From a correlation of drill records and seismic data over the entire project, the range of velocities for the various types of overburden was determined. In

the areas investigated, velocities of 1,000 to 2,000 ft per sec generally indicated very loose material; velocities of 3,500 ft to 5,000 ft per sec usually were indicative of relatively soft material such as silt or clay; but in some instances velocities from 4,000 to 5,000 ft per sec did indicate fairly loose deposits of glacial till. Velocities in excess of 5,000 ft per sec usually indicated the presence of compact glacial tills.

In a large proportion of the test locations, two types or conditions of overburden exist as shown in Fig. 4. On the till ridges, velocities usually ranged from 1,000 to 2,000 ft per sec, to depths of from 5 to 8 ft. This zone of loose material is indicated by the steep portion of the graph (Fig. 4) through the origin. Beneath this zone of loose material, the velocity usually changed to a much higher value, the magnitude of which depended on the character of the material. Along the margin of the river and also in the bed of the river, overburden of a single velocity was often found. In some locations, three types of overburden were found, as shown in Fig. 5:

Soil type	Feet
Upper soil, H_1	5.1
Clay, H_2	12.5
Till, H_3	71.7
Depth to rock	89.3

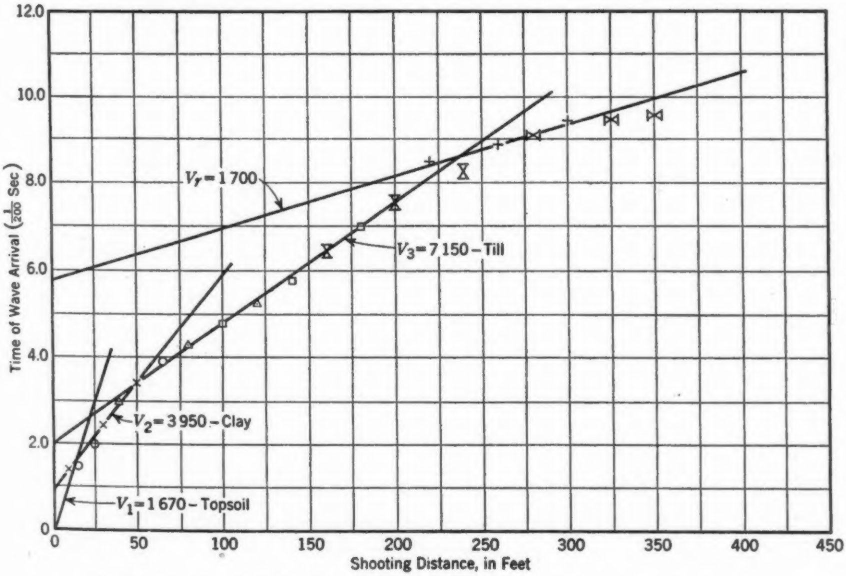


FIG. 5.—TYPICAL TIME-DISTANCE GRAPH SHOWING THREE TYPES OF OVERBURDEN

The soil profile as indicated by such graphs may not always be recognized in drill logs or by visual inspection, as velocity characteristics are determined by the depth of frost action, compaction, and other factors which may not appear from sampling.

Failure to Identify Shallow Rock.—In one section of the Point Rockway Canal area difficulty was encountered in identifying, properly, the hard rock that is within a few feet of the ground surface. In the analysis of the observed data, the intermediate material (V_2 = 5,000 ft per sec) was believed to be clay or till, although the irregularity of the data threw some doubt on this interpretation, and indicated the possible presence of a zone of seamed or fractured rock. Upon checking with the drill, relatively sound cores were obtained in this zone. A comparatively low velocity of 5,000 ft per sec for shallow and fractured rock is not uncommon in other formations but because of the unusually sound character of the rock previously encountered in the St. Lawrence River investigations, this condition was not anticipated. The low velocity was probably the result of seams opened by frost action and the absence of a heavy overburden load. Under a heavy overburden it is believed that this rock would have exhibited a much higher velocity.

To further investigate and study this irregularity found in the Point Rockway Canal, experimental seismic tests were made in the vicinity of the Northern Quarries near Norfolk, N. Y. At the quarries, where the rock face was exposed, a fractured and weathered zone could be seen to a depth of several feet. This rock was under a shallow overburden and was of the same origin as that in the Rockway Canal. The seismic tests adjacent to the quarries produced graphs very similar to those obtained from the data in the canal and showed that, under some conditions of fracturing, very shallow rock cannot always be identified by the seismic method.

Effect of Frozen Ground.—As early as the middle of December, 1940, frost in the ground began to affect results in some areas, and by February it had

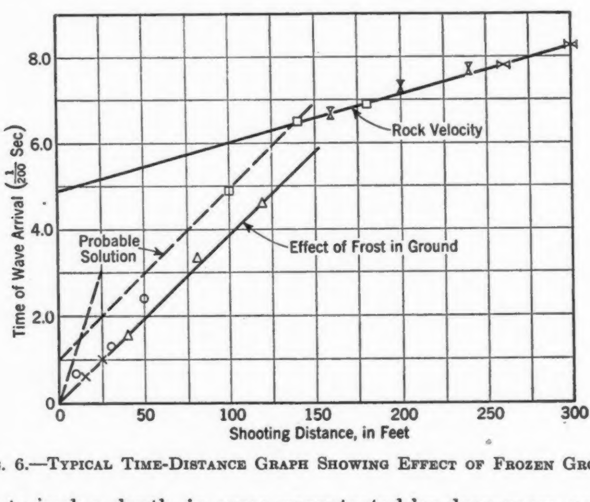


FIG. 6.—TYPICAL TIME-DISTANCE GRAPH SHOWING EFFECT OF FROZEN GROUND

penetrated to such a depth, in areas unprotected by deep snow, as to interfere seriously with the accuracy of interpretations. Accordingly, field work was suspended during March and April. As the velocity of wave propagation in frozen soil is much higher than that in normal or unfrozen upper soil, this

condition leads to uncertainties in the interpretation of seismic data. The effect of 6 in. of frozen ground is shown in Fig. 6. Velocities of 5,000 to 7,000 ft per sec, values abnormally high for upper soil, were obtained for shooting distances as great as 100 ft. For longer shooting distances the crust of frozen ground did not carry sufficient energy to register on the film. Probable solu-

Fig. 7.—Waterproof Detector Housings

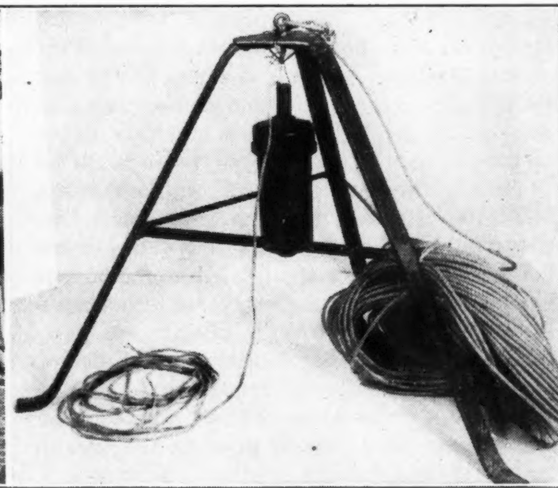


Fig. 8.—Tripod Detector Mounting for Use in Deep Water

tions were developed for many of the lines affected by frost. These were believed to be reasonably accurate, in regard to total depth of overburden, but their graphs are of little value for indicating the soil profile or character of overburden. Unfortunately many of the lines affected by frost were in areas of deep overburden where a knowledge of the overburden properties was of more importance than the depth to rock.

Subaqueous Exploration

Tests in Quiet or Slowly Moving Water.—The need for information on the character and depth of overburden in areas of the river channel where the water was quiet, or was moving slowly, called for the development of special equipment and a revised field procedure for under-water exploration. One of the essential requirements in making seismic tests on the bed of a river or other bodies of water is to provide means for placing detectors and shots with reasonable accuracy at predetermined locations. Another requirement is to waterproof the detectors and so mount them that they will assume a vertical position when lowered to the bed of the stream.

To place the shots and detectors accurately a float or marker line was constructed by attaching wooden blocks at designated points on a rope approximately 600 ft long. Measured from the center of the line, floats were attached at the following points: 0, 10, 40, 80, 130, 180, and 230 ft ahead and 0, 25, 40,

80, 130, 180, and 230 ft back. The center float, and those at 40 ft on either side of the center, marked the detector positions, and the other blocks marked the shot points as on a normal land line.

One end of the float line was anchored on the bed of the river at the upper end of the seismic line along which information was desired. A heavy stone usually served for this purpose. The line was then allowed to float downstream until it assumed a stable position. Where the current was not sufficient to stretch the rope, both ends were anchored after maneuvering the line into the desired position. In some instances it was necessary to anchor the center of the line also to prevent it from drifting with the wind or current. Buoys were attached at anchor points, when necessary, to keep the markers afloat. Empty oil cans of 2-gal or 5-gal capacity were found satisfactory for this purpose.

Two methods were used to keep the detectors in a vertical position on the bed of the stream. For depths of water not in excess of 10 ft the mounting shown in Fig. 7 was used. Each detector was provided with a waterproof housing made from a 10-in length of 3.5-in. pipe. The lower end was closed with a steel plate and the upper end with a cap containing a ¾-in. pipe nipple. In assembling the system the detector was placed in the housing and a pair of rubber covered auxiliary leads about 30 ft long were attached to the terminals. Cotton waste was packed around the sides of the detector and over the terminals, after which the leads were passed through the nipple and the cap tightly screwed on. It is important, of course, that the detector rest firmly on the bottom of the housing.

After stretching and anchoring the float line the three detectors were set at designated points. A pipe or rod was driven at each detector position and the detector assembly lashed to it in a vertical position. A boat containing the

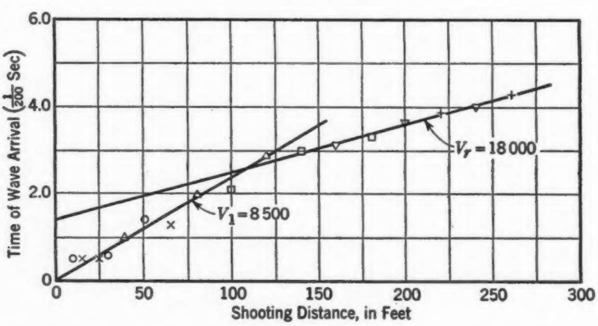


FIG. 9.—TYPICAL TIME-DISTANCE GRAPH OBTAINED WITH THE TRIPOD DETECTOR MOUNTING UNDER WATER

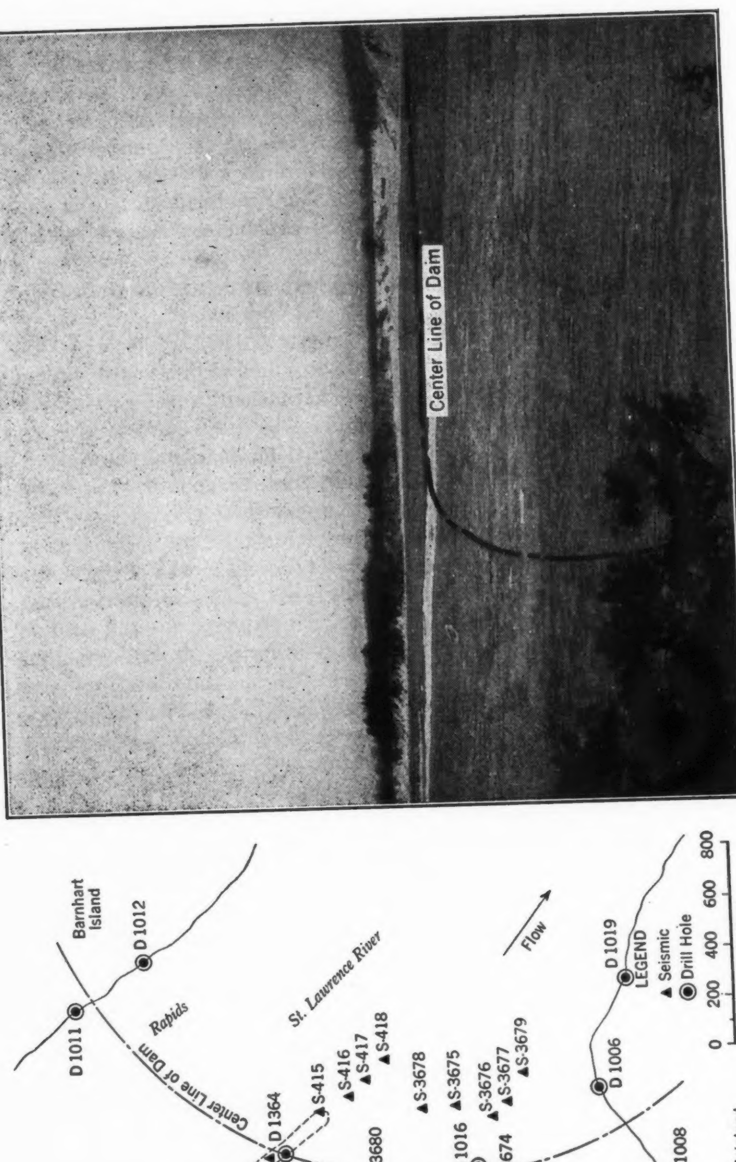
receiving apparatus was anchored about 50 ft from the center of the line and the three pairs of detector leads used in land work were connected to the auxiliary leads. A rock was tied to a piece of flagging or burlap to act as a sinker for the charge, and the cap wires were spliced to the firing line, which, in turn, terminated at the receiving instrument. The charge was then lowered to the bottom, after which it was detonated as in land practice.

Where the water was more than 10 ft deep, the detector, in the same waterproof housing as that described, was suspended from the apex of a heavy iron tripod, as shown in Fig. 8. After sealing the nipple against the entrance of water the entire assembly was lowered to the bed of the river by a rope. The detector assumed a vertical position irrespective of the configuration of the river bed. Although it did not rest directly on the bottom it received the shock from the explosion through the legs of the tripod and the suspension wire. No difficulty was encountered in obtaining satisfactory records with either type of detector mounting.

A typical graph obtained by under-water exploration is shown in Fig. 9, the depth to sound rock in this case being $H = \frac{1.4 \times 8{,}500}{400 \times 0.88} = 33.8$ ft. This is an excellent graph and typical of most of those obtained in quiet or slowly moving water. In fact, the results are considered more accurate than those obtained on land, since, due to erosion, the overburden was more uniform in character, and usually a two-layer graph was obtained. The main source of error compared with the procedure used on land is in the placement of the charges in deep or moving water. It was found that under these conditions, the charge might drift a considerable distance before reaching the bottom, and allowance had to be made for this effect. Large errors in the placement of the shot, especially for the shorter shooting distances, will result in inaccurate time-distance graphs. Fortunately, where allowances had to be made for the drift of the charges, the overburden conditions were quite uniform so that the results from the shorter shooting distances were not needed for accurate interpretation.

Tests in Swift Water.—Seismic tests were made in the river channel area at the Long Sault Dam site where swift water made drilling too expensive to be included in the present program of exploration. The Long Sault Dam site is located at the foot of the Long Sault Rapids and the proposed dam site extends from the New York mainland to the head of Barnhart Island. Drillings, as shown in Fig. 10(*a*), were made on the shores of the mainland, on Barnhart and Long Sault islands, in the south channel where floats could be used, and on the shoal in the middle of the channel. The information obtained by drilling was very limited along the full extent of the axis of the dam. Soundings in the river channel made several years ago indicated that a rock gorge might exist at the site, especially between the shoal and Barnhart Island. As further drilling operations along the proposed axis of the dam would require large floating equipment and would be very expensive, seismic explorations were made wherever possible. It was found impracticable to make tests between the shoal and Barnhart Island where the velocity of the river is approximately 12 miles per hr and power boats cannot be maneuvered satisfactorily. A limited number of tests were made below the shoal and in the vicinity of the axis of the dam between the shoal and the mainland where the velocity of the river is not so great. Fig. 10(*b*) is a photograph of the river, taken from the New York mainland looking toward Barnhart Island, which shows the condition of the water in the area investigated.

In undertaking seismic exploration in the channel area it was realized that the usual accuracy could not be obtained and that the exploration would not



(a) Location of Drillings

(b) View of the Site

FIG. 10.—LONG SAULT DAM SITE

cover the area between the shoal and Barnhart Island. Nevertheless, it was believed that results could be obtained which would give the approximate elevation and configuration of the bedrock in the area explored, and even indicate the trend of the rock floor in the unexplored area. As it was impossible to place detectors on the river bed in excessively swift water, it was necessary to develop a procedure different from that used in quiet water. Fig. 11 illustrates

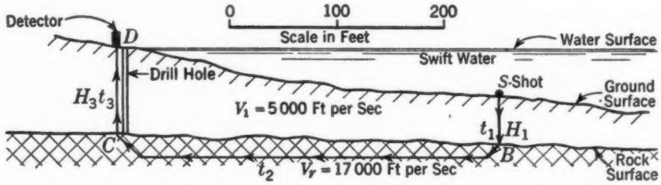


FIG. 11.—ARRANGEMENT OF APPARATUS FOR DETERMINING DEPTH OF OVERBURDEN IN SWIFT WATER

the arrangement of apparatus used in these tests. A single detector and the oscillograph recorder were placed on shore close to the region to be explored. The detector was placed near a drill hole or seismic line where the depth to rock was known. The velocity in the adjacent overburden was determined in the usual manner. Single charges of dynamite were then placed with a float or boat at points on the river bottom and fired as in land practice.

One group of shots (S-3674 to S-3680, inclusive) were placed and fired while the detector was at drill hole D-1059. The locations of these shots are shown in Fig. 10(*a*). The firing line and a heavy sinker were attached to the charge which was then carried out in the river and dropped. The location of the charge, when dropped, was determined by two observers with transits. If possible, soundings were taken at the time of dropping the charge; otherwise the approximate elevation of the river bottom was determined from a map showing the subaqueous contours in the vicinity. The application of this method was limited, owing to the difficulty of handling the firing line in swift water.

Another group of single shots, S-415 to S-418, inclusive (also shown in Fig. 10(*a*)) were placed and fired while the detector was on the shoal in the channel, 120 ft from drill hole D-1364 and at the center of the line S-414. The firing line and a heavy sinker were attached to the charge which was then placed on a float made of two oil drums lashed together. The float attached to a cable was allowed to drift downstream with the current but was controlled by a winch mounted on the shoal. To keep the firing line from fouling, it was threaded through rings on the cable. When the float was at the desired location, a boat came alongside and the charge was released and dropped into the river. The location of the charge was determined by observers on shore. The float was then pulled 50 to 100 ft upstream and the shot fired. The depth of the river at the location of the shot was determined by actual soundings or from the map showing the subaqueous contours.

The method of analyzing the data obtained in the swift water at the Long Sault Dam site differed from the usual method of analysis as the knowns and unknowns were different and the necessary assumptions more radical. Refer-

ring to Fig. 12, the only information obtained from the film record was the total time, T, which elapsed from the shot instant to the arrival of the shock at the detector, D, Fig. 11. This total time T includes: t_1, the time of transit through the overburden at the shot S; t_2, the time of transit through the rock from B to C, Fig. 11; and t_3, the time of transit through the overburden CD. The value T was plotted on the time-distance coordinates at Q for the shooting distance OR, as shown in Fig. 12. As the velocity in the rock was known from previous tests on shore, it was possible to draw the line QM which gives PM or t_2 as the time

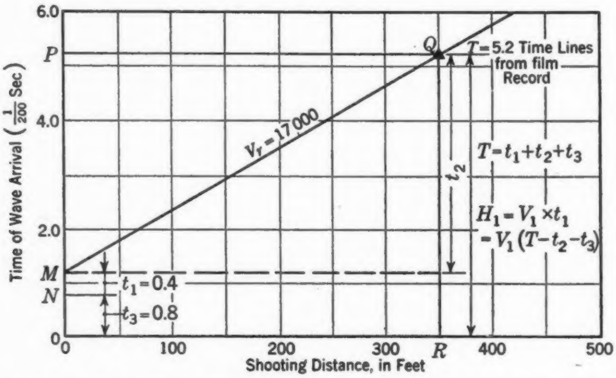


FIG. 12.—ANALYSIS OF OBSERVATIONS FOR DETERMINING DEPTH OF OVERBURDEN IN SWIFT WATER

of transit in the rock. As the depth CD was known from boring and seismic records, and the velocity, V_1, of wave propagation in the overburden from seismic tests on land, it was possible to draw line ON as t_3, the time of transit in the overburden at CD. This left MN, or t_1, as the time consumed in the overburden SB. To compute the depth H, or SB, it was necessary to assume the same overburden velocity there as at CD.

The results, as obtained from shooting in swift water, contain certain probable inherent inaccuracies. The assumption that the velocities in the overburden and rock in the river channel are the same as those determined on the adjacent land could be in error. However, any error resulting from this assumption will be only in proportion to velocity variations from place to place in the respective media. A more likely source of error is that in obtaining the true value of t_1. Where the depths to be measured are not great, the value of t_1 is small in comparison with T, and any errors in determining t_2 and t_3 will be carried into t_1 and may seriously affect its accuracy. Another source of error is in the determination of the shooting distance. With the velocity ratio of 17,000 to 5,000 ft per sec for rock and overburden as shown, an error of 3.4 ft in the shooting distance will result in a 1-ft error in H_1. The accuracy in determining the elevation of the rock also depends on the accuracy of the sounding of the river bottom. For large values of H_1 these errors are relatively small, but where attempts are made to measure shallow overburdens in this manner the indications may be entirely misleading. The data and conditions of the tests in the rapids have been analyzed and studied carefully and, although

limited in accuracy as to the actual thickness of the relatively shallow overburden in the bed of the river, it is believed that the results are reliable as indicating the trend of the rock contours and in precluding the possibility of a deep gorge in the area explored.

Correlation of Seismic and Drilling Results

The correlation between seismic information and drilling data has been thoroughly investigated over the entire project. Table 1 contains the seismic

TABLE 1.—Comparison of Seismic Interpretations with Drilling Records, St. Lawrence River Project

Seismic line No.	Location	Drill hole	Distance from drill hole to seismic center (ft)	Seismic			Drill Hole		
				Ground elevation	Depth (ft)	Rock elevation	Rock elevation	Depth (ft)	Ground elevation
S-2	Long Sault Dam......	D-1019	20	174.8	29.7	145.1	144.0	28.5	172.5
S-11	Long Sault Dam......	D-1007	70	212.7	56.6	156.1	160.1	53.0	213.1
S-12	Long Sault Dam......	D-1007	120	212.5	52.0	160.5	160.1	53.0	213.1
S-34	Power house..........	D-1029	57	185.6	43.6	142.0	146.9	38.7	185.6
S-35	Power house..........	D-1136	90	191.9	58.7	133.2	132.5	60.5	193.0
S-38	New Cornwall Canal...	D-1121	140	211.8	47.4	164.4	162.6	49.0	211.6
S-50	Iroquois Dam.........	D-1046	27	231.5	25.0	206.5	206.4	25.9	232.3
S-66	Grass River Lock......	D-1368	100	184.0	74.3	109.7	104.4	77.5	181.9
S-68	Robinson Bay Lock....	D-1070	126	199.7	65.2	134.5	138.2	61.8	200.0
S-74	Ogden Island..........	D-1083	116	232.1	53.0	179.1	183.5	50.6	234.1
S-88	Rockway Canal.......	D-1264	125	228.5	26.8	201.7	202.1	24.8	226.9
S-89	Point Rockway Canal..	D-1073	115	229.5	30.0	199.5	198.1	34.2	232.3
S-158	Massena Canal Intake Works..............	D-1260	43	218.4	41.6	176.8	171.3	47.3	218.6
S-164	Point Rockway Canal..	D-1281	39	228.0	21.1	206.9	200.5	27.0	227.5
S-175	Point Rockway Canal..	D-1283	89	241.1	9.0	232.1	233.8	4.1	237.9
S-203	Galop Island Channel..	D-1110	0	247.7	41.3	206.4	205.4	42.3	247.7
S-207	Lalone Island Channel.	D-1102	60	236.5	15.0	221.5	221.8	26.0	247.8
S-225	Galop Island Channel..	D-1109	173	272.5	75.0	197.5	198.6	70.6	269.2
S-246	Galop Island Channel..	D-1106	0	250.2	52.5	197.7	196.2	54.0	250.2
S-299	Iroquois Dam.........	D-1045	100	233.5	31.6	201.9	208.1	23.5	231.6
S-318	Point Rockway Canal..	D-1341	0	240.1	30.4	209.7	209.6	30.5	240.1
S-320	Point Rockway Canal..	D-1379	0	237.1	4.5[a] 14.6[b]	232.6 222.5	232.6	5.0	237.6
S-323	Point Rockway Canal..	D-1343	0	244.6	4.3[a] 20.7[b]	240.3 223.9	243.8	0.8	244.6
S-325	Point Rockway Canal..	D-1378	40	233.2	3.7[a] 6.7[b]	279.5 226.5	228.4	4.3	232.7
S-408	Long Sault Island.....	D-1391	150	249.0	72.4	176.6	172.4	83.3	255.7
S-412	Long Sault Island.....	D-1390	0	257.5	77.5	180.0	168.9	88.6	257.5

[a] To top of fractured rock. [b] To top of sound rock.

and drilling data at all locations where a drill hole was close enough to the center of a seismic line for a significant comparison to be made. In a few locations drill holes were spotted at or near the centers of seismic lines for the express purpose of checking the seismic predictions. This is true for lines 203 and 246 on Galop Island, lines 318, 320, 323, and 325 in the Rockway Canal area, and line 412 on Long Sault Island. On Galop Island the checks were remarkably close, being 1 ft in a depth of 42.3 ft and 1.5 ft in a depth of 54.0 ft, respectively. At line 412 there was a discrepancy of 11 ft in a total depth of 88.6 ft, which is believed to be the result of a very compact overburden condition immediately above the bedrock surface. Correlations in the Rockway Canal area were not entirely satisfactory, owing to the conditions previously described. At line 269, near the Massena Power Canal intake, there was an error of 9.9 ft in a

depth of 67.2 ft. Failure at this location to determine the depth of rock more accurately was the result of an artificial fill of varying depth created by the dumping of spoil from the excavation of the power canal. It should be noted that in all three of these locations, where the correlations are not satisfactory, the drillings were made to check apparent inconsistencies in the seismic data. With the exception of lines 269, 412, and a few of those in the Rockway Canal area, the correlations in Table 1 are quite satisfactory and indeed much better than obtained ordinarily in geophysical explorations.

The differences in depths of overburden, as determined by the two methods, are with few exceptions within the limit of the relief in the rock surface. For the greater depths the differences are relatively small, while for the shallower overburdens—such as at lines 89, 164, 175, 318, and 325—the actual differences, although only a few feet, may represent a large percentage of error. It should be remembered, however, that in all cases the drill record gives the depth of the overburden at one precise location, whereas the seismograph records an average depth over a considerable length of line.

Significance of Investigations

Heretofore the seismic method of exploration usually has been considered primarily applicable for making preliminary or reconnaissance investigations and has not been used extensively for making more detailed analyses. As a guide to drilling and for obtaining comparative data it has been of considerable value on alternate dam and spillway sites investigated by the U. S. Corps of Engineers. After a site for a structure has been definitely selected, it is customary to obtain detailed information by drilling. On the St. Lawrence River Project, the seismic data have been used more extensively than is generally feasible. All proposed cuts and channels were explored primarily by seismic methods, a few drill holes having been added to determine overburden conditions and to verify seismic results where bedrock occurred above or near excavation grade. The results have been used to a large extent in fixing locations and grades for the proposed cuts and channels in an effort to minimize the excavation of ledge rock. Other sites were explored with a small number of seismic lines to supplement the drilling program.

The use of the seismic data has enabled the design of concrete structures, navigation channels and hydraulic cuts to proceed more rapidly under the present program than if exploration consisted only of drill holes. A total of 355 satisfactory seismic determinations, and 47 of fair reliability, were made on land and in quiet water. To make the same number of rock elevation determinations by drilling, it would have been necessary to drill approximately 15,600 ft. The cost of such drilling would have been about four times that of the seismic investigations, an amount too great to have been included in the exploration program. Also the cost of additional drilling in the channel at the Long Sault Dam site would have been prohibitive. Although seismic tests could not be conducted over this entire swift-water area, the few results obtained were useful for design and estimate purposes.

Seismic determinations are valuable depending upon their accuracy in comparison to that required for the study of each feature. The accuracy, in turn,

depends upon correct interpretation of the data obtained in the field. Reliable results can be obtained only by an experienced interpreter who understands the geological and surface conditions affecting the data. In general, the subsurface conditions were ideal for the seismic method and the results could be used with greater assurance, therefore, than in areas where conditions are more variable. The results of the seismic and drilling exploration were plotted on the various site maps, after which rock-surface contours were drawn. Although only one rock determination was computed for each line, the graphs often could be used to indicate the direction of slope of the bedrock surface. The results of the determinations at Long Sault Dam site, although not considered as accurate as other measurements, definitely indicated that if a bedrock gorge does exist, it is confined to the narrow area between the shoal and Barnhart Island.

The results of the investigation were very useful in interpreting overburden conditions, particularly in the areas of channels, cuts, and canals. By correlation with drill-hole data and the general geology of the region, the results aided in determining the extent and depth of various deposits of glacial till and other types of overburden and also their condition of density and compaction.

The subaqueous tests are believed to be unique for obtaining information of the type required for this project. Reliable results were obtained where detectors could be placed on the river bottom and sufficiently accurate results for required purposes were obtained in part of the channel area at the Long Sault Dam site where the water is swift. As the results of subaqueous exploration were satisfactory on this project, the use of the seismic method should be broadened in future investigations to include similar conditions.

Acknowledgments

The seismic investigations were made under the direction of Col. A. B. Jones, M. Am. Soc. C. E., district engineer of the St. Lawrence River District. Credit is due to F. P. Fifer, M. Am. Soc. C. E., head engineer on the project, for helpful and encouraging advice, particularly in regard to the subaqueous tests; A. E. Wood, associate geologist in the Binghamton District, supervised the work and analyzed the data during the early period of the investigation; G. R. Butterfield, Jr., Jun. Am. Soc. C. E., assistant engineer, rendered valuable assistance in laying out the seismic program and correlating it with the drilling program, and also in the final preparation of this paper; W. A. Wells assisted throughout the investigation in the interpretation and analysis of the field data and also in the preparation of the final report.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

PENDLETON LEVEE FAILURE

By Kenneth E. Fields,[1] and William L. Wells,[2]
Assoc. Members, Am. Soc. C. E.

Synopsis

In the fall of 1939 the Vicksburg Engineer District was forced to construct a setback levee along the south bank of the Arkansas River near Pendleton, Ark. At one point the proposed levee, known as the Pendleton New Levee, crosses Lake Lenox, a shallow body of water which was formerly the channel of the Arkansas River. Foundation investigations revealed that the levee would be underlain by extremely soft clay at the lake crossing. Calculations of sections required to insure stability showed that the most economical procedure would be to displace the clay. In view of this fact, construction of the levee presented an excellent opportunity for securing data concerning a foundation failure.

Investigations of the foundation were made to determine soil conditions and characteristics. Various devices were installed in the foundation to observe movements, and hydrostatic and earth pressures. By means of such observations, it was thought that valuable and much-needed data concerning the physics of the failure would be obtained. The data obtained are given herein along with several analyses of the failure employing these data.

Field and Laboratory Data

The results of the foundation explorations are shown in Fig. 1. As can be noted, foundation conditions were found to be as follows:

(*a*) A thin, noncontinuous surface stratum of clay (not shown);
(*b*) A stratum of fine sand, from 4 ft to 9 ft thick;
(*c*) A stratum of extremely soft clay, averaging about 12 ft thick; and
(*d*) A relatively thick stratum of fine to coarse sand, underlying (*c*).

The usual physical soil tests were performed on undisturbed samples of the soft clay. From consideration of the age of the deposit and the time-consolida-

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by **May 1, 1943.**

[1] Lt. Col., Corps of Engrs., U. S. Army, Camp Polk, La.

[2] Pres., Vicksburg Bridge Co., Vicksburg, Miss.

tion characteristics of the material, it was found that the foundation material was fully consolidated under its present overburden pressure. From the results of conventional direct-shear tests of the consolidated quick type, a value of cohesion of 0.04 ton per sq ft was obtained, as well as an angle of internal friction

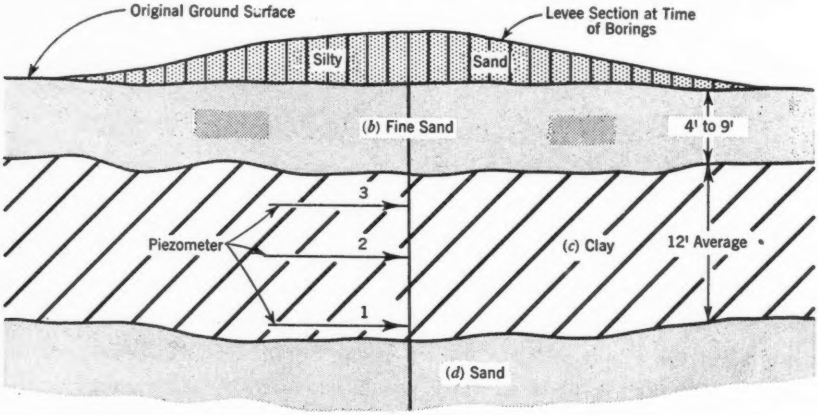


FIG. 1.—GENERALIZED FOUNDATION CROSS SECTION

of 19° (see Fig. 2). With these data, stability computations were made to determine the cross section that would produce failure for a height of approximately 30 ft.

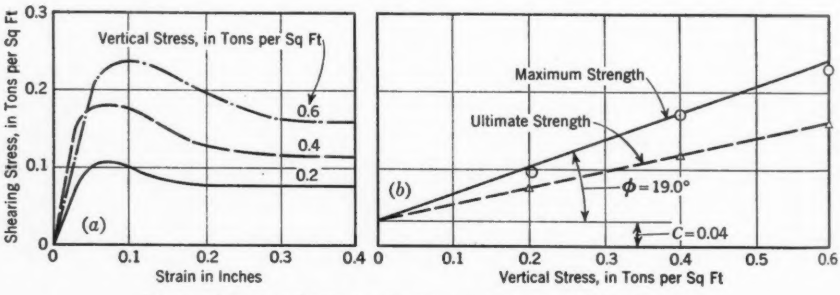


FIG. 2.—SHEARING-STRENGTH CURVES; SAMPLE FROM CLAY STRATUM

PROVISIONS FOR PHYSICAL MEASUREMENTS

Provisions for physical measurements consisted of settlement plates, earth and hydrostatic pressure cells, and cross-section surveys. Settlement plates were of the usual type (see Fig. 3(*a*)). Because of the short time available, the only earth pressure cells which could be obtained were six Wilson cells. These are similar to the Goldbeck cell in that internal air pressure is increased until a balance with the external pressure is reached. Further details of the installation are of no importance inasmuch as the cells failed to function.

As a means of observing pore-water pressures in the foundation during construction, piezometers were installed in the underlying clay stratum at one cross section. Details of the apparatus are shown in Fig. 3(*b*).

Construction and Failure

Construction consisted first of the formation of the base of the embankment, the material for which was hauled in by tractors and wagons and spread by

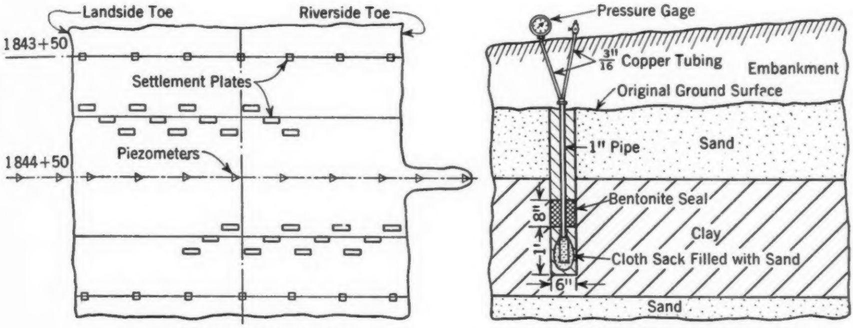


Fig. 3.—Instrument Layout

Fig. 4.—General View of Failure

bulldozers. After the completion of the base, on January 15, 1939, a triangular section having a base width of about 320 ft and a total height of about 18 ft was then built. From this section a triangular section was built in 3-ft lifts as rapidly as possible.

On the night of February 14, 1940, when the section had reached a height of about 32 ft, the foundation failed. This failure was confined to the land-side

half of the embankment and extended longitudinally for about 500 ft. Fig. 4 shows a part of the embankment after failure. As can be noted from this photograph and from Fig. 5, the following phenomena were characteristic of the failure:

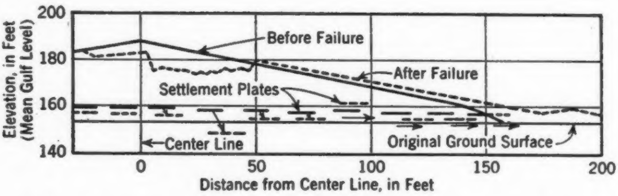


Fig. 5.—Movements of Settlement Plates; Land Side of Levee

(*a*) A nearly vertical subsidence of 6 to 10 ft of that part of the embankment between the center line and points 50 to 70 ft land side of the center line (this subsidence left a trough-shaped depression in the embankment, in plan curving toward the land-side toe near the ends of the failure zone);

(*b*) A horizontal movement outward of the remainder of the land-side part of the embankment (approximately 10 ft); and

(*c*) An upheaval, 4 to 5 ft high, just beyond the land-side toe.

The most significant physical data obtained were those concerning pore pressures. Readings of the piezometer gages to the nearest 0.25 lb per sq in. were taken daily during the period of construction and to failure and beyond, except on days when the site was inaccessible due to rains. Fig. 6 shows plots of several typical sets of pore pressure data and corresponding loading diagrams. To be noted is the correlation between hydrostatic excess pressure and structure pressure near the center line and the building up of hydrostatic excess pressure to values greater than the pressure imposed by the structure near the toes (175 ft from the center line). Fig. 7 contains hydrostatic excess pressure data for all stations on the several dates indicated.

Notation

The letter symbols in this paper conform essentially with *Manual of Engineering Practice No. 22*.

Analyses of Failure

In the study of the failure, plots, such as the one in Fig. 8, were made for each piezometer station. Thus, at Station 1844 + 50, 100 ft to the land side, the original effective vertical stress (the overburden load) was apparently decreased at piezometers 1 and 2. The word "apparently" is used purposely because the total stresses were estimated by use of the elastic theory and actual values may be different. If these effective stresses are used in the following well-known shearing strength formula:

$$s = c + (p - u) \tan \phi \qquad (1)$$

a loss of strength is indicated at all such locations where effective stresses decreased. Such decrease occurred at all stations near the toes of the structure and particularly near the bottom of the clay stratum. (In Eq. 1: s = unit shearing strength; c = cohesion per unit area; p = total normal stress on the

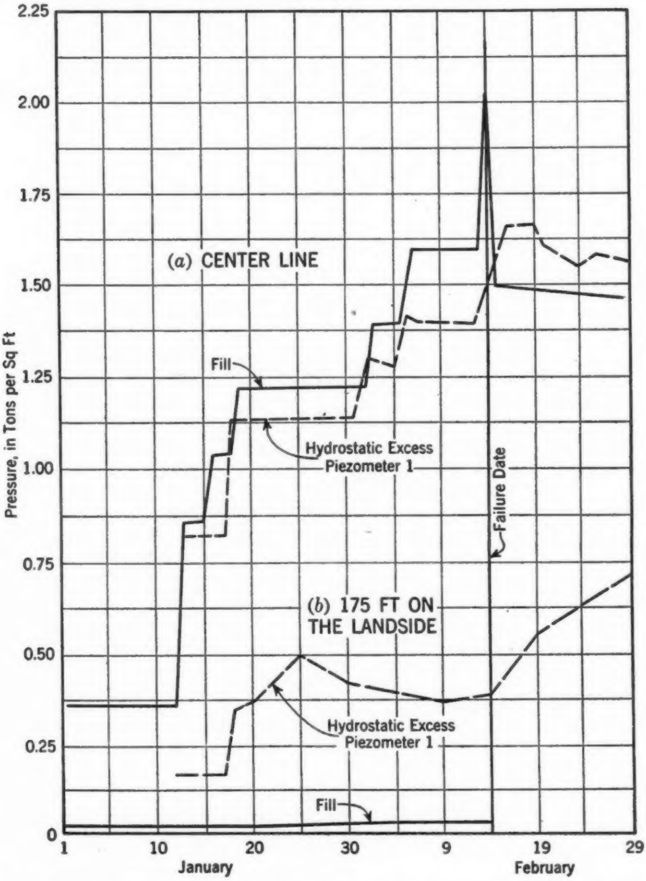


FIG. 6.—HYDROSTATIC EXCESS AND FILL PRESSURES, STATION 1844 + 50, DURING 1940

plane of failure; u = unit hydrostatic or pore water pressure; and ϕ = true angle of internal friction of the soil.) Fig. 9 shows plots of shearing strength along the bottom of the clay stratum, estimated by use of Eq. 1. (The apparent increase in strength on the river side between February 5 and February 14 was caused by an unexplained decrease in the reading of piezometer 1, 100 ft to the river side at this station.)

To extend the analysis of the failure still further in the foregoing manner, wherein shearing strength is expressed by Eq. 1, stability analyses by a modified circular arc method were made of the land side of the levee—the side that failed.

(See the Appendix for a description of the modified circular arc method.) These analyses at several cross sections in the failure zone gave factors of safety on the day of failure varying from 0.90 to 1.07. The factor of safety at one lift

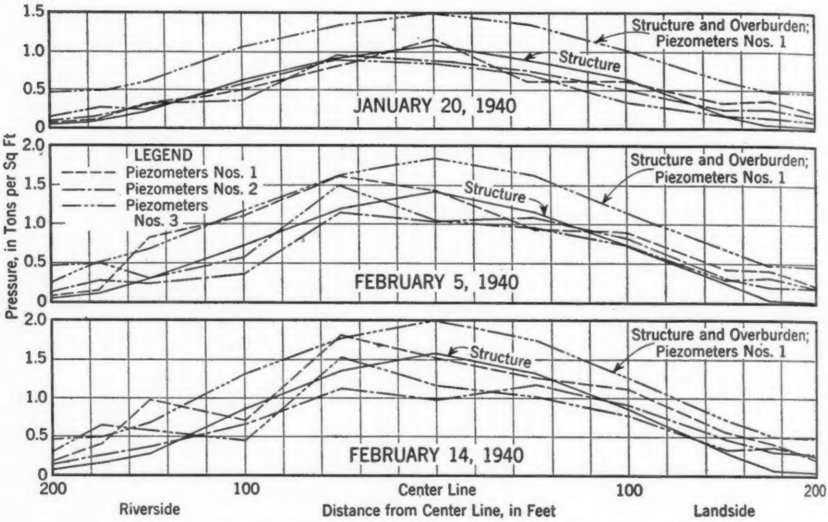


FIG. 7.—STRUCTURE AND EXCESS HYDROSTATIC PRESSURES, STATION 1844 + 50

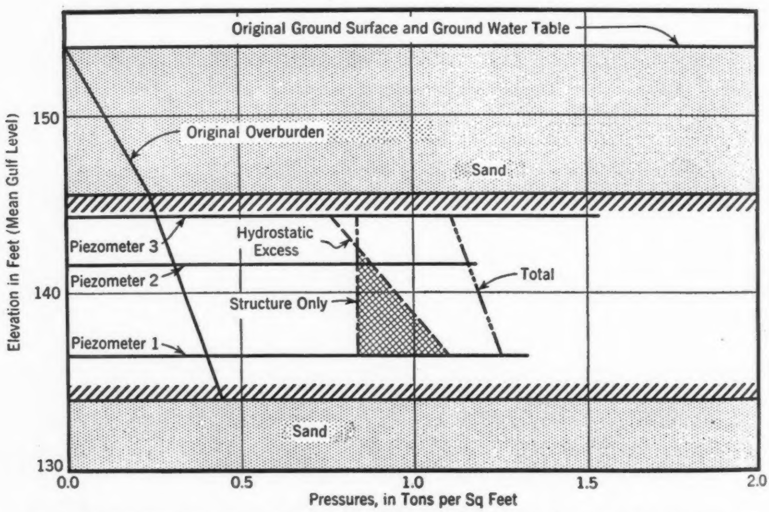


FIG. 8.—STRUCTURE AND HYDROSTATIC EXCESS PRESSURES, 100 FT ON THE LAND SIDE AT STATION 1844 + 50, FEBRUARY 14, 1940

before failure at Station 1843 + 50 was 1.31; just prior to failure, 0.90; and the day after failure, 0.98. Thus this method appears reliable when the failure surface and the hydrostatic excess pressures are employed in the stability

analyses together with values of shearing strength, as expressed by Eq. 1, and values of c and ϕ, as determined by consolidated quick tests.

The peculiarity of the foregoing method of computation is that the values of shearing strength employed are based upon consolidated quick test results. This is at variance with the present belief that when effective stresses are used in the computation of shearing strength, slow test results must be used to determine c-values and ϕ-values. This belief is held because the slow test is run slowly enough to prevent a build-up of pore-water stress, and thus the stresses applied to the specimen are effective stresses. The shear curve resulting from a number of slow tests is based on effective stresses. In the consolidated quick test, however, the test is run rapidly and pore pressures built up. Thus, the applied stresses are not effective stresses but total stresses—part pore-water pressure, part effective stress—and the resulting shear curve is based on total stresses.

It was found that if estimated in the same manner as the land side, the river-side should have failed sooner than the land side. Unfortunately, the exploration and testing programs were not extensive enough to provide sufficient data to determine whether this was due to greater strength in parts of the river-side foundation; or whether the method of analysis used on the land side resulted in a factor of safety of approximately 1.0 only by accident—and that actually the method is not correct. This doubt, together with the fact that c-values and ϕ-values were determined from consolidated quick tests rather than slow tests, suggests caution in other cases; and also the great need for continued study of other failures and of the factors determining shearing strength.

A more recent interpretation of the data obtained on the failure has been one in which the pore pressure data have been used to indicate the change in distribution of total stresses rather than as a means of estimating effective stresses.

The analysis is based on the premise that the excess pore pressures were built up primarily by the increased stress or deformation caused by the applied load. As the failure data approached, some of the excess pore pressures were no doubt caused by the remolding of the clay in areas where shearing stress exceeded strength. However, disregarding such a factor for the moment, it is of interest to consider the pore pressures only as an indication of the change in stress distribution which occurred; that is, from them to estimate total stresses on the day of failure. If this is done then (see Figs. 10 and 11) at Station 1844 + 50, on the 175-ft land side, where upheaval occurred, the total stresses on the cross-hatched, rectangular section (Fig. 11) of the clay stratum prior to construction were, for practical purposes: σ_y (average) = 0.38 ton per sq ft; and σ_x (average) = 0.34 ton per sq ft. (The assumption is made that $\sigma_x = 0.90\ \sigma_y$ for pressure at rest.) Values in Fig. 11 were computed from data in Fig. 10. From the pore pressure data the increase in pore pressure was 0.35 ton per sq ft (average). Considering this to mean an equal increase in σ_x, and that σ_y remained unchanged, since there is sand above and below the clay stratum, the total stresses on the section on the day of failure were: $\sigma_y = 0.41$ ton per sq ft (there was an increase of applied load of 0.03 ton per sq ft); and $\sigma_x = 0.37 + 0.35 = 0.72$ ton per sq ft. The maximum shearing stress, τ (max), for these princi-

pal stresses is then: $\tau\ (\max) = \frac{\sigma_1 - \sigma_3}{2} = \frac{(0.37 + 0.35) - 0.41}{2} = 0.16$ ton per sq ft. This shearing stress at failure checks quite well with the shearing strength as indicated by the consolidated quick test data, since, for a normal pressure of $\sigma_y = 0.38$ (the original average overburden pressure for the section), the shear-

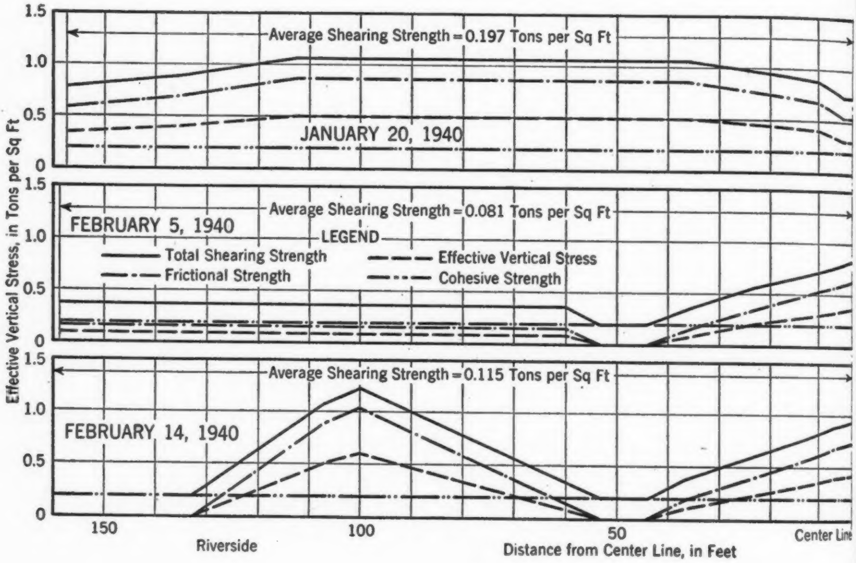


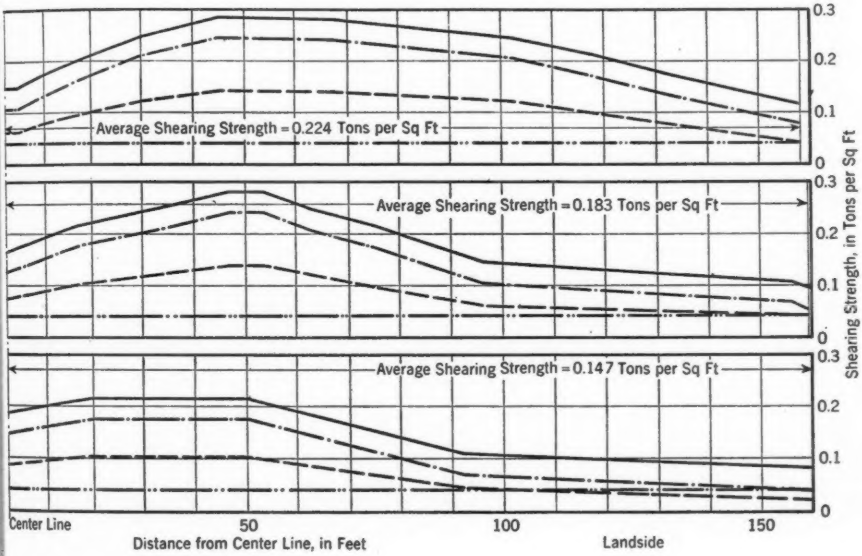
FIG. 9.—SHEARING STRENGTH ALONG THE BOTTOM

ing strength is 0.17 ton per sq ft (see Fig. 2). Similar computations for the river side also indicate a fairly close agreement of stress and strength.

Another interesting interpretation of the failure is that it was caused by the building up of pore pressure in the clay stratum near the toes of the structure to values equal to or greater than the overburden pressure at that point. Thus the upper piezometer located 175 ft on land side of the center line (Fig. 10), where upheaval occurred, indicated a hydrostatic excess pressure of 0.36 ton per sq ft on the day of failure. Computed vertical pressure, due to the natural overburden and structure, at the same point and on the same date was 0.35 ton per sq ft. It is pertinent to note that similar comparisons for the river side show that the hydrostatic excess pressure exceeded the computed vertical stress prior to the date of actual failure.

The natural question is how to proceed in subsequent problems. In an effort to indicate how an engineer would proceed in other problems, a stability analysis by the modified circular arc method for the slope and height attained was made, using the values of strength corresponding to original overburden pressures and the consolidated quick test curve. Although this computation gave a factor of safety of 1.3, failure occurred for that cross section. This may be due to the fact that in such an analysis the engineer assumes that all along

the failure arc the maximum shearing strength is mobilized. Since the materials vary in strength-deformation characteristics, the strain necessary to mobilize maximum strength will vary. Failure will be progressive and in the upper sections such as segments 1A and 2A, Fig. 14, Appendix, cracks may occur prior to complete failure. These decrease the shearing strength there. Hence it is



OF THE CLAY STRATUM; STATION 1844 + 50

normally impossible to mobilize maximum strength at all points at the same time. By referring to Fig. 12(*c*) (Appendix), which shows the forces on a slice of the embankment, it can be seen that, to maintain equilibrium, either shearing strength on the bottom of the slice or normal force on the side of the slice must be mobilized. For equilibrium a decrease or deficiency in one must be balanced by an increase or sufficiency in the other. At Pendleton the σ_x-forces at the toes were built up to their limiting values when the factor of safety, estimated on the original strength, equaled 1.3. Thus, the maximum strength along the failure arc was not mobilized at the time of failure. How much of this lack of maximum strength is due to insufficient strain, or to failure and remolding of the clay, cannot be determined from the available data. All that can be said is that such lack of maximum strength apparently did exist and that it is probably common in every failure.

The writers believe that this latter picture of the physics of the failure is the correct one. Thus, the visualization of the problem is one having two aspects. The first, in design, is the use of strengths from quick tests and a factor of safety which will insure that sufficient shearing strength is mobilized to prevent pressures at the toe sufficient to cause upheaval. The factor of safety to use would depend upon the materials involved, and their characteristics. The second, in

control of construction, is the measurement of total pressures and pore pressures to prevent upheaval at the toes.

Of course, more definite recommendations for future cases are desirable; but the shearing strength of clays is a subject on which there is much more

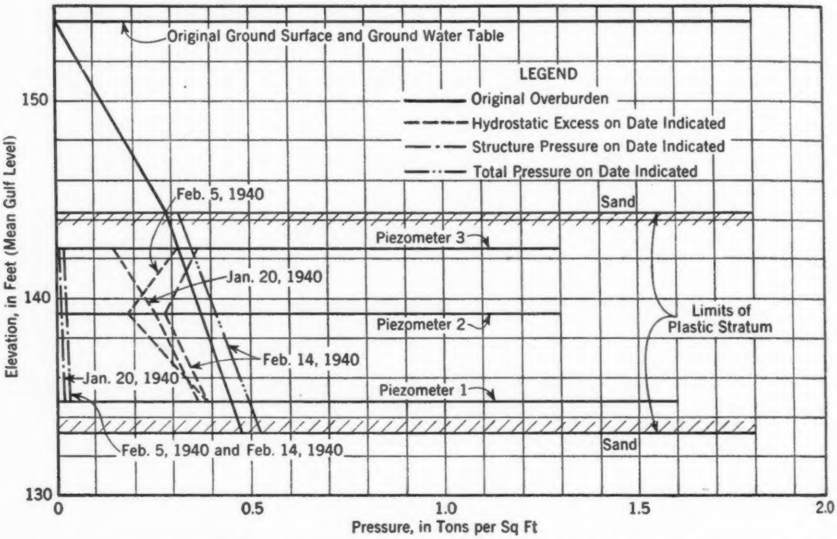


FIG. 10.—STRUCTURE AND HYDROSTATIC PRESSURES, 175 FT ON THE LAND SIDE, AT STATION 1844 + 50

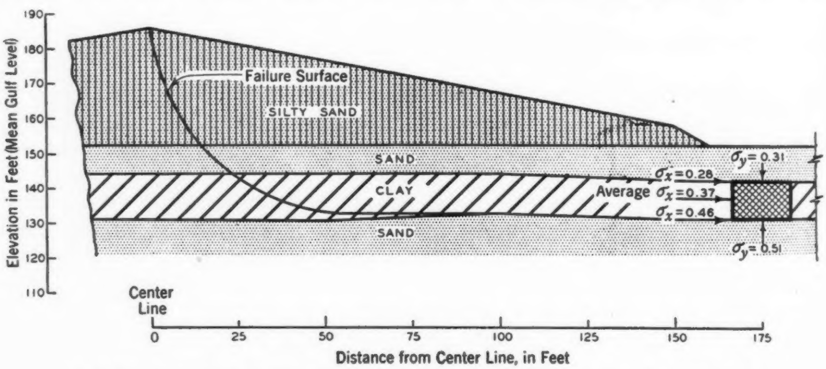


FIG. 11.—STABILITY OF THE TOE SECTION, 175 FT ON THE LAND SIDE, AT STATION 1844 + 50, FEBRUARY 14, 1940
(All Stresses in Tons Per Square Foot)

to learn. The trend of all thought has been toward the belief that effective stresses are a prime factor. To date, however, there have been few, if any, cases wherein the effect of effective stresses on shearing strength has been isolated. Measurements of both pore pressures and total pressures must be accomplished to do this. It is believed that the Pendleton experiences will assist, in some measure, the future studies of this most difficult question.

APPENDIX

Modified Circular Arc Method

The modified circular arc method differs from the normal method in regard to the resultant forces considered to be acting on the failure surfaces of the slices

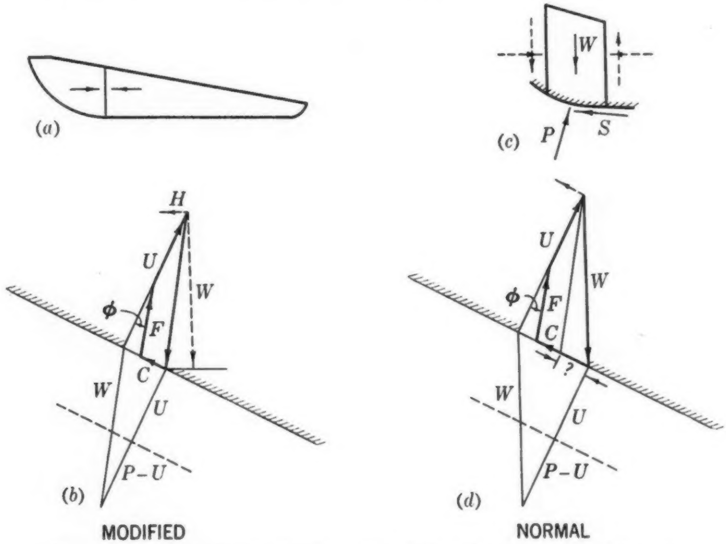


Fig. 12.—Comparison of Modified and Normal Circular Arc Methods

into which the sliding mass is divided. In the normal method, the forces acting on the sides of the slices are neglected since they are assumed to be equal and opposite. Actually, because of equilibrium requirements, such an assumption means that the slice is considered to exert a force on the failure surface which is the resultant of the weight of the material in the slice and a balancing force acting parallel to the failure surface (see Fig. 12). In the modified method the force is considered to be the resultant of the weight of the material in the slice and a balancing force acting horizontally. Fig. 12(*a*) shows that in the Pendleton failure the movement of the greater part of the sliding mass was horizontal.

TABLE 1.—Characteristics of Materials

Material (see Fig. 13)	Material	Weight (lb per cu ft)	Cohesion (tons per sq ft)	Angle of internal friction φ
A	Embankment material...	112	0.10	32.5°
B	Foundation sand........	120	0.0	32.5°
C	Foundation clay........	96	0.04	19.0°

The modified circular arc method is shown in greater detail in Fig. 13 with Tables 1, 2, and 3. The forces, shown in Fig. 14, are:

(*a*) The total weight of the segment, W (soil plus water), known in magnitude and direction;

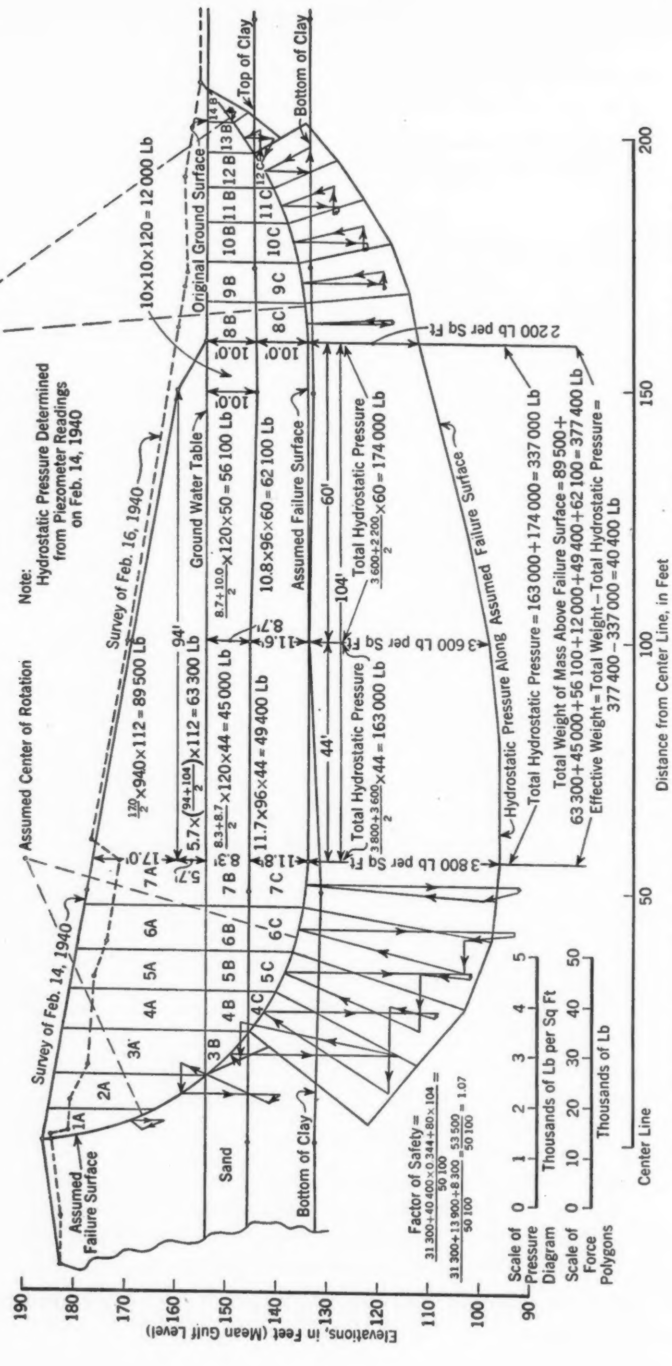


FIG. 13.—STABILITY ANALYSIS OF THE PENDLETON NEW LEVEE (LAND SIDE), BY THE MODIFIED CIRCULAR ARC METHOD

TABLE 2.—VALUES OF u AND c

Segment No. (see Fig. 13)	(a) HYDROSTATIC PRESSURES					(b) TOTAL COHESION		
	Left	Right	Average	Length of chord (ft)	Total force (lb)	Length of arc (ft)	Cohesion	
							Tons	Lb
1	....	....	....	....	....	22.4	2.24	4,480
2	....	....	....	....	....	13.0	1.30	2,600
3	0.0	520	260	12.4	3,220	12.6	....	....
4	3,000	3,200	3,100	9.6	29,800	9.7	0.39	780
5	3,200	3,600	3,400	8.6	29,300	8.7	0.35	700
6	3,600	3,800	3,700	9.2	34,000	9.2	0.37	740
7	3,800	3,800	3,800	9.2	35,000	9.2	0.37	740
8	2,200	2,100	2,150	8.0	17,200	8.0	0.32	640
9	2,100	1,900	2,000	8.2	16,400	8.2	0.33	660
10	1,900	1,600	1,750	8.2	14,400	8.3	0.33	660
11	1,600	1,400	1,500	7.5	11,300	7.7	0.31	620
12	1,400	1,200	1,300	8.0	10,400	8.0	0.32	640
13	600	350	475	7.3	3,470	7.3	....	....
14	350	0	175	8.7	1,520	8.8	....	....

TABLE 3.—COMPUTATION OF WEIGHTS OF SEGMENTS

Segment No.[a] (Fig. 13)	Width (ft)	HEIGHT, IN FEET			Area (sq ft)	Weight (lb)	H (lb)
		Left	Right	Average			
1A	6.0	0.0	20.3	10.1	60.6	6,800	0
2A	7.0	20.3	30.0	25.1	175.8	19,700	6,200
3A	9.0	30.0	28.2	29.1	26.2	29,400	....
3B	9.0	0.0	8.4	4.2	37.8	4,540	....
Total						33,940	8,400
4A	8.0	28.2	26.8	27.5	220.0	24,600	....
4B	8.0	8.4	8.4	8.4	67.2	8,050	....
4C	8.0	0.0	5.2	2.6	20.8	2,000	....
Total						34,650	17,200
5A	8.0	26.8	25.6	26.2	210.0	23,500	....
5B	8.0	8.4	8.4	8.4	67.2	8,050	....
5C	8.0	5.2	9.7	7.4	59.2	5,690	....
Total						37,240	11,800
6A	9.0	25.6	24.2	24.9	224.0	25,100	....
6B	9.0	8.4	8.4	8.4	75.5	9,050	....
6C	9.0	9.7	10.8	10.2	91.6	8,800	....
Total						42,950	6,500
7A	9.0	24.2	22.7	23.4	211.0	23,600	....
7B	9.0	8.4	8.3	8.3	74.7	8,950	....
7C	9.0	10.8	11.8	11.3	102.0	9,800	....
Total						42,350	0
8B	8.0	10.0	9.8	9.9	79.2	9,500	....
8C	8.0	10.0	9.7	9.9	79.2	7,600	....
Total						17,100	−1,500
9B	8.0	9.8	9.8	9.8	78.3	9,390	....
9C	8.0	9.7	8.3	9.0	72.0	6,910	....
Total						16,300	−3,300
10B	8.0	9.8	9.7	9.7	77.6	9,300	....
10C	8.0	8.3	6.2	7.2	57.6	5,540	....
Total						14,840	−5,200
11B	7.0	9.7	9.7	9.7	67.9	8,140	....
11C	7.0	6.2	3.6	4.9	34.4	3,310	....
Total						11,450	−5,900
12B	7.0	9.7	9.7	9.7	68.0	8,150	....
12C	7.0	3.6	0.0	1.8	12.6	1,210	....
Total						9,360	−5,800
13B	6.0	9.7	5.5	7.6	45.6	5,460	−7,900
14B	6.7	5.5	0.0	2.7	18.1	1,740	−2,200

[a] A signifies embankment material; B signifies foundation sand; and C signifies clay.

Total positive 50,100
Total negative 31,300

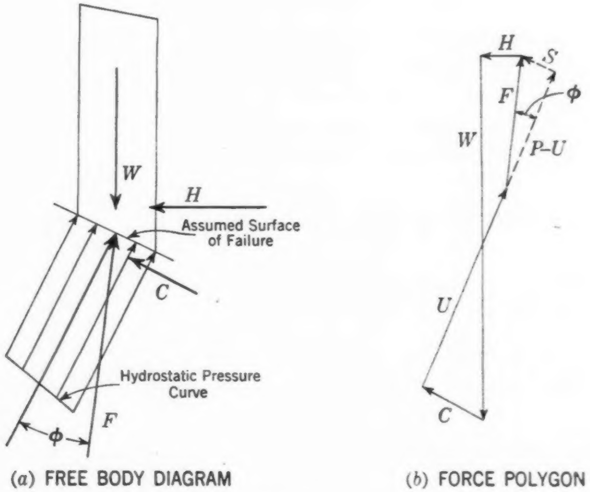
(*b*) The total cohesion, *C*, acting along the base of the slice, known in magnitude and direction;

(*c*) The total hydrostatic pressure, *U*, acting on the base of the slice (the direction of this force is normal to the assumed failure surface and its magnitude is determined from the hydrostatic pressure observations);

(*d*) The intergranular soil reaction, *F*, acting on the base of the slice (the direction of this force is at an obliquity of ϕ degrees with the normal to the assumed failure surface);

(*e*) The total normal stress, *P*; and

(*f*) The horizontal force, *H*, required either to maintain the slice in equilibrium or to overcome the resistance of the slice to movement (its direction is assumed to be horizontal, as movement of the greater portion of the sliding mass was horizontal).



(*a*) FREE BODY DIAGRAM (*b*) FORCE POLYGON

Fig. 14

Items (*a*) and (*b*) were determined for each slice from consideration of the dimensions of the slice and the soil tests of the materials composing the slice. Force (*c*) was determined from the plot of hydrostatic pressure distribution contained in Fig. 7. These forces were plotted in magnitude and direction, as shown in Fig. 14. The direction of the soil reaction $(p - u)$, was then plotted on the force diagrams, and the magnitude of the horizontal force to close each diagram was determined. Horizontal forces acting in the direction of motion were considered positive, whereas those acting to oppose motion were considered as negative.

The factor of safety is then defined as the ratio of the horizontal forces opposing motion to the unbalanced forces tending to cause motion. This is not comparable to the factor of safety as ordinarily defined (that is, the ratio of total resisting forces to total driving forces), but is a more sensitive index of stability as unbalanced forces only are considered.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

AERATION OF SPILLWAYS

By G. H. Hickox,[1] M. Am. Soc. C. E.

Synopsis

A method of computing the size of air vent needed for aeration of spillways, based on all of the available data, is presented in this paper. The effect of insufficient aeration on the reduction of pressure beneath the nappe and on the discharge is discussed. Formulas and diagrams for calculation of these effects are included.

Introduction

The aeration of spillways and measuring weirs has apparently not been given the attention it deserves. The direct result of insufficient aeration of overfall weirs is a reduction of pressure beneath the nappe due to the removal of air by the overfalling jet. This reduction of pressure in turn causes several undesirable secondary effects. The first is an increase of pressure difference on the weir itself. The reduction of pressure on the downstream side may increase the resultant load on the structure to the point of failure. In the case of dam A, spillway discharge flows over a gate 20 ft high. This gate was designed for a water load 20 ft over its crest, or 37,400 lb per ft of length. During tests on the aeration of this spillway, negative pressures under the nappe of 9.7 ft of water were measured. This was equivalent to an additional load of 12,100 lb per ft of length, or an increase of 32%. The significance of this increase in load becomes more apparent when it is noted that this was by no means the maximum pressure reduction that could have been obtained by closing the aeration vents completely. In this particular case, it represented the maximum extra load which it was thought the gate could support with safety.

The second effect is a change in the appearance of the nappe. Because the difference in pressure acts across the nappe itself, the nappe is depressed below its normal position. In the case of large spillways, the upper surface becomes ridged in the direction of flow and begins to entrain air instead of being smooth as for normal flow conditions. For thin nappes and considerable

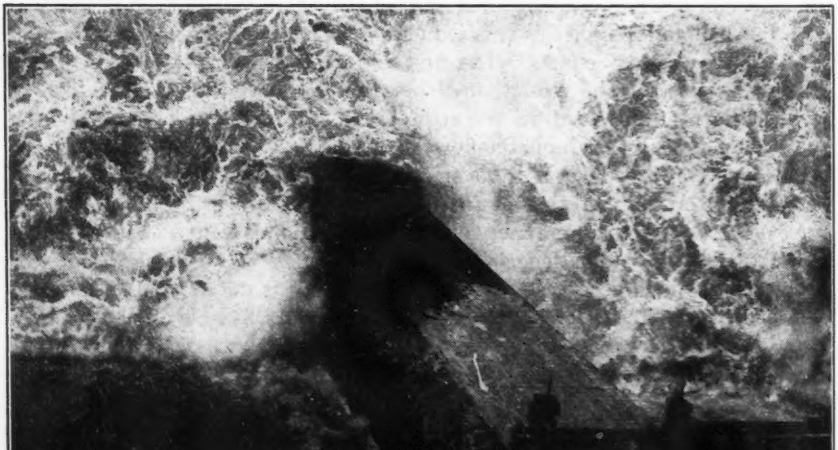
Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by May 1, 1943.

[1] (Deleted by censors.)

pressure reduction, the nappe may break, allowing air to pass through it. As soon as this occurs, the nappe springs back to its normal position and the process of air exhaustion and breaking of the nappe is repeated. Depression of the nappe because of insufficient aeration is illustrated on a small scale model by Figs. 1(*a*) and 1(*b*) and for a large spillway by Fig. 1(*c*).

(*a*) Discharge Over Model (1 : 25 Scale) of Lower Gate Leaf; Full (or Complete) Aeration

(*b*) Discharge Over Model (1 : 25 Scale) of Lower Gate Leaf; No Aeration



(*c*) Prototype: Gate at Left Aerated; 2-Ft Vacuum Under Nappe at Right

FIG. 1.—DEPRESSION OF THE NAPPE BECAUSE OF INSUFFICIENT AERATION, DAM A

A third result of the reduction of pressure beneath the nappe is an increase in the pressure difference causing flow over the weir, and a corresponding increase in the discharge coefficient. For a given head, the discharge is increased. If the discharge is fixed, the measured head is reduced.

Reduction of pressure beneath a spillway nappe is also undesirable from the standpoint of the performance of hydraulic models. Fig. 1(*b*) illustrates a pressure reduction of about 1.5 ft in a 1 : 25 scale model, or 37 ft in the prototype; a result which is clearly not possible. In some cases, the performance of a stilling basin may be completely altered because of the difference in the manner in which the spillway discharge enters it, due entirely to lack of aeration of the nappe.

Where a weir is to be used for measuring purposes, lack of aeration results in an increased discharge, and sometimes, as mentioned, a fluctuation or pulsation of the nappe, which may be very objectionable.

There is a definite need for information on the effect of lack of aeration of weirs. Such information is necessary (*a*) to prevent excessive loads on spillway gates; (*b*) to insure the reliability of model tests, with respect both to the pressure reduction to be expected in the prototype, and the performance of the stilling basin; and (*c*) to provide a criterion for the necessary aeration of measuring weirs.

Nomenclature

The letter symbols used in this paper are defined where they first appear in the paper and are assembled for reference in the Appendix.

Experimental Data

The problem of aeration became important with the construction of dam A. There are twenty-two spillway gates at this dam, each built in two sections 20 ft by 40 ft, operating in the same vertical plane. Discharge is regulated by first lifting the upper leaves and allowing the water to flow over the tops of the lower leaves. The lower leaves are not raised until after all the upper leaves are open. The design of the piers is such that self-aeration is impossible and aeration vents are necessary. The same gate design and similar operation were provided for dams B, C, and D. At dam D, the problem was complicated by the fact that, under certain conditions, water might discharge beneath the lower gates as well as above them. A knowledge of the size of aeration vents required was necessary in designing these projects,

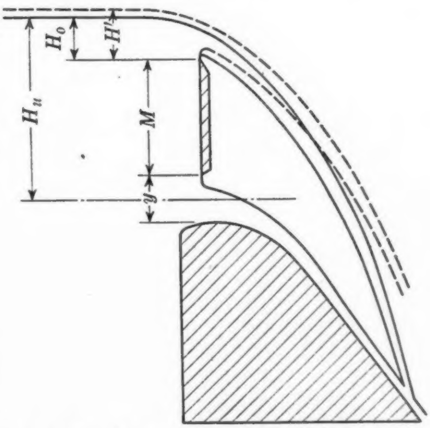


Fig. 2.—Experimental Setup, Schematic Drawing Showing Dimensions Measured

TABLE 1.—AERATION OF A SHARP-CRESTED WEIR (VARIOUS STRUCTURES)

Test No.	Feet		Cubic Feet per Second			p (ft)[b]	$\frac{H_o}{H'}$	$\frac{p}{H_o}$	α	β
	H'	H_o	Q_o	q_o	q_a					
(1)	(2)	(3)	(4)	(5)	(6)[a]	(7)	(8)	(9)	(10)	(11)
	(a) DATA REPORTED BY JOE W. JOHNSON, ASSOC. M. AM. SOC. C. E.									
1	0.306	0.295	0.59	....	401	0.0672	0.964	0.228	....	....
2	0.295	0.284	0.56	....	415	0.0695	0.963	0.245	....	....
3	0.276	0.268	0.506	....	351	0.0588	0.971	0.219	....	....
4	0.252	0.246	0.44	....	271	0.0454	0.976	0.185	....	....
5	0.222	0.219	0.361	....	131	0.0219	0.986	0.100	....	....
6	0.306	0.297	0.59	....	367	0.0615	0.970	0.207	....	....
7	0.273	0.267	0.498	....	245	0.0411	0.978	0.154	....	....
8	0.245	0.239	0.420	....	208	0.0349	0.975	0.146	....	....
9	0.228	0.225	0.376	....	157	0.0272	0.987	0.121	....	....
14	0.255	0.250	0.446	....	176	0.0295	0.981	0.118	....	....
18	0.306	0.299	0.59	....	390	0.0653	0.977	0.218	....	....
19	0.306	0.297	0.59	....	458	0.0769	0.970	0.259	....	....
20	0.306	0.302	0.59	....	294	0.0482	0.986	0.160	....	....
22	0.284	0.276	0.528	....	332	0.055	0.972	0.199	....	....
23	0.284	0.277	0.528	....	263	0.044	0.975	0.159	....	....
24	0.284	0.277	0.528	....	247	0.0413	0.975	0.149	....	....
25	0.284	0.281	0.528	....	93	0.0156	0.990	0.056	....	....
26	0.253	0.247	0.441	....	224	0.0375	0.976	0.152	....	....
27	0.253	0.247	0.441	....	180	0.0301	0.976	0.122	....	....
28	0.253	0.249	0.441	....	131	0.0221	0.985	0.089	....	....
30	0.239	0.235	0.404	....	145	0.0243	0.983	0.103	....	....
31	0.239	0.235	0.404	....	118	0.0198	0.983	0.084	....	....
32	0.239	0.237	0.404	....	81	0.0135	0.992	0.057	....	....
33	0.205	0.201	0.320	....	40	0.0335	0.980	0.167	....	....
36	0.307	0.299	0.59	....	304	0.051	0.975	0.171	....	....
37	0.307	0.300	0.59	....	244	0.041	0.978	0.137	....	....
38	0.307	0.304	0.59	....	102	0.017	0.990	0.056	....	....
44	0.308	0.299	0.59	....	441	0.074	0.971	0.248	....	....
45	0.308	0.300	0.59	....	364	0.061	0.974	0.204	....	....
46	0.308	0.301	0.59	....	312	0.052	0.977	0.173	....	....
47	0.308	0.305	0.59	....	96	0.0162	0.990	0.053	....	....
48	0.284	0.276	0.528	....	380	0.0635	0.971	0.230	....	....
49	0.284	0.276	0.528	....	297	0.0488	0.971	0.177	....	....
50	0.284	0.276	0.528	....	250	0.0424	0.971	0.153	....	....
51	0.284	0.279	0.528	....	104	0.0174	0.982	0.062	....	....
52	0.260	0.252	0.46	....	344	0.0576	0.970	0.229	....	....
53	0.260	0.253	0.46	....	260	0.0434	0.973	0.172	....	....
54	0.260	0.255	0.46	....	185	0.0309	0.981	0.121	....	....
55	0.227	0.223	0.376	....	138	0.023	0.983	0.103	....	....
56	0.227	0.224	0.376	....	120	0.020	0.987	0.089	....	....
57	0.227	0.223	0.376	....	84	0.014	0.983	0.063	....	....
58	0.227	0.224	0.376	....	44	0.0073	0.987	0.033	....	....
59	0.193	0.191	0.297	....	73	0.0122	0.990	0.064	....	....
60	0.193	0.191	0.297	....	60	0.0099	0.990	0.052	....	....
61	0.193	0.192	0.297	....	42	0.0071	0.995	0.037	....	....
62	0.308	0.295	0.597	....	577	0.0966	0.957	0.328	....	....
63	0.308	0.296	0.597	....	490	0.082	0.961	0.277	....	....
64	0.308	0.299	0.597	....	330	0.0552	0.970	0.185	....	....
65	0.308	0.297	0.597	....	502	0.084	0.964	0.283	....	....
66	0.308	0.299	0.597	....	435	0.0728	0.970	0.244	....	....
67	0.308	0.300	0.597	....	310	0.0515	0.974	0.172	....	....
68	0.284	0.272	0.528	....	440	0.0740	0.958	0.272	....	....
69	0.202	0.196	0.313	....	51	0.0422	0.970	0.215	....	....
70	0.202	0.201	0.312	....	21	0.0174	0.995	0.087	....	....
71	0.202	0.200	0.312	....	19	0.0157	0.990	0.079	....	....
72	0.202	0.198	0.312	....	14	0.0118	0.980	0.060	....	....
73	0.175	0.171	0.253	....	33	0.0271	0.976	0.159	....	....
74	0.313	0.304	0.61	....	337	0.0564	0.971	0.186	....	....
75	0.286	0.279	0.534	....	274	0.0458	0.976	0.164	....	....

TABLE 1.—(*Continued*)

Test No.	Feet		Cubic Feet per Second			p (ft)[b]	$\frac{H_o}{H'}$	$\frac{p}{H_o}$	α	β
	H'	H_o	Q_o	q_o	q_a					
(1)	(2)	(3)	(4)	(5)	(6)[a]	(7)	(8)	(9)	(10)	(11)
	(*a*) Data Reported by Joe W. Johnson, Assoc. M. Am. Soc. C. E.—*Cont.*									
76	0.243	0.239	0.414	....	192	0.0321	0.984	0.134	....	....
83	0.312	0.299	0.608	....	918	0.093	0.958	0.311	0.0774	0.249
84	0.286	0.276	0.533	....	775	0.078	0.965	0.283	0.0801	0.284
85	0.259	0.254	0.459	....	400	0.041	0.981	0.161	0.0736	0.456
86	0.226	0.222[c]	0.371	....	220	0.022	0.984	0.100	0.0725	0.732
87	0.188	0.185[c]	0.281	....	24	0.019	0.984	0.103	0.0299	0.291
88	0.199	0.196	0.306	....	21	0.0176	0.985	0.0898	0.0270	0.301
89	0.217	0.213	0.349	....	27	0.0225	0.982	0.106	0.0264	0.250
90	0.240	0.235	0.408	....	49	0.040	0.979	0.170	0.0279	0.164
91	0.253	0.246	0.441	....	63	0.052	0.973	0.211	0.0284	0.134
92	0.269	0.263	0.486	....	240	0.040	0.978	0.152	0.0551	0.363
93	0.292	0.282	0.551	....	420	0.070	0.966	0.248	0.0596	0.240
94	0.312	0.302[c]	0.607	....	690	0.07	0.967	0.232	0.0712	0.307
95	0.290	0.282	0.545	....	530	0.053	0.972	0.188	0.0717	0.381
96	0.256	0.251[c]	0.449	....	310	0.032	0.981	0.127	0.0697	0.547
97	0.223	0.220[c]	0.364	....	170	0.017	0.987	0.0773	0.0686	0.888
98	0.223	0.220[c]	0.364	....	206	0.021	0.985	0.0955	0.0723	0.757
99	0.126	0.125[c]	0.156	....	46	0.0078	0.989	0.0624	0.0769	1.23
100	0.166	0.163[c]	0.234	....	110	0.018	0.983	0.110	0.0742	0.672
101	0.184	0.180[c]	0.273	....	135	0.023	0.981	0.128	0.0695	0.544
102	0.204	0.200[c]	0.318	....	150	0.025	0.981	0.125	0.0645	0.516
103	0.249	0.242[c]	0.43	....	300	0.051	0.970	0.210	0.0632	0.300
104	0.268	0.261[c]	0.483	....	475	0.048	0.973	0.184	0.0751	0.408
105	0.285	0.277[c]	0.53	....	535	0.054	0.972	0.195	0.0729	0.374
106	0.311	0.300[c]	0.605	....	715	0.072	0.966	0.240	0.0727	0.303
	(*b*) Eight-Foot Weir in Hydraulic Laboratory									
1	0.904	0.874	....	2.88	2.72	0.202	0.967	0.231	0.0375	0.163
2	1.003	0.952	....	3.38	3.40	0.322	0.949	0.338	0.0341	0.101
3	0.939	0.902	....	3.05	2.99	0.246	0.961	0.273	0.0361	0.133
4	0.717	0.700	....	2.03	2.05	0.113	0.977	0.161	0.0469	0.290
5	0.602	0.589	....	1.562	1.63	0.0701	0.979	0.119	0.0559	0.469
6	0.547	0.543	....	1.357	1.27	0.0428	0.992	0.0788	0.0605	0.767
7	0.483	0.478	....	1.128	0.965	0.0246	0.990	0.0515	0.0689	1.34
8	0.328	0.327	....	0.636	0.682	0.0123	0.997	0.0376	0.101	2.68
9	1.060	0.970	....	3.68	3.93	0.440	0.915	0.454	0.0333	0.0735
10	0.813	0.787	....	2.45	2.09	0.118	0.968	0.150	0.0417	0.278
11	0.894	0.867	....	2.83	2.19	0.130	0.970	0.150	0.0378	0.252
12	1.038	1.012	....	3.56	10.03	0.171	0.975	0.169	0.0645	0.382
13	0.970	0.960	....	3.21	5.71	0.0538	0.990	0.0560	0.0688	1.23
14	0.826	0.811	....	2.51	7.04	0.0816	0.982	0.101	0.0814	0.809
15	0.676	0.668	....	1.860	5.13	0.0434	0.988	0.0650	0.0988	1.52
16	0.434	0.430	....	0.962	3.32	0.0182	0.991	0.0424	0.153	3.62
17	0.815	0.803	....	2.47	6.45	0.0686	0.985	0.0855	0.0822	0.962
18	0.942	0.924	....	3.07	9.74	0.161	0.981	0.174	0.0708	0.406
19	1.012	0.984	....	3.43	9.94	0.168	0.968	0.170	0.0666	0.390
20	0.258	0.258	....	0.446	1.03	0.00174	1.000	0.00674	0.0256	37.9
21	0.725	0.724	....	2.07	32.8	0.00896	0.998	0.0124	0.342	27.6
22	0.837	0.836	....	2.57	43.6	0.0159	0.999	0.0190	0.296	15.5
23	0.884	0.880	....	2.79	49.3	0.0203	0.996	0.0231	0.281	12.2
24	0.994	0.990	....	3.33	52.0	0.0226	0.996	0.0228	0.250	10.9
25	1.097	1.086	....	3.88	71.6	0.0429	0.990	0.0395	0.228	5.78
26	0.386	0.387	....	0.809	32.0	0.0086	0.997	0.0223	0.635	28.6
27	0.815	0.814	....	2.47	30.8	0.0079	0.999	0.0097	0.304	31.2
28	0.942	0.942	....	3.07	38.6	0.0125	1.000	0.0133	0.262	19.8
29	1.012	1.011	....	3.43	51.6	0.0222	0.999	0.0219	0.244	11.1
30	0.258	0.258	....	0.446	14.3	0.00171	1.000	0.00663	0.958	14.4

TABLE 1.—(*Continued*)

Test No. (1)	FEET: H' (2)	FEET: H_o (3)	CUBIC FEET PER SECOND: Q_o (4)	CUBIC FEET PER SECOND: q_o (5)	CUBIC FEET PER SECOND: q_a (6)[a]	p (ft)[b] (7)	$\frac{H_o}{H'}$ (8)	$\frac{p}{H_o}$ (9)	α (10)	β (11)
(*b*) EIGHT-FOOT WEIR IN HYDRAULIC LABORATORY—*Cont.*										
31	0.860	0.841	....	2.56	....	0.109	0.978	0.130	....	....
32	0.797	0.783	....	2.38	....	0.0903	0.982	0.115	....	....
33	0.698	0.686	....	1.95	....	0.0608	0.983	0.0887	....	....
34	0.605	0.597	....	1.58	....	0.0382	0.988	0.0636	....	....
35	0.498	0.494	....	1.18	....	0.0156	0.992	0.0316	....	....
36	0.408	0.406	....	0.870	....	0.0121	0.995	0.0298	....	....
37	0.269	0.267	....	0.474	....	0.0104	0.993	0.0390	....	....
38	0.920	0.899	....	2.96	....	0.141	0.977	0.157	....	....
39	0.983	0.953	....	3.28	....	0.188	0.970	0.197	....	....
40	1.062	1.019	....	3.68	....	0.290	0.958	0.285	....	....
41	0.815	0.781	....	2.46	....	0.175	0.959	0.224	....	....
42	0.942	0.910	....	3.07	....	0.352	0.966	0.387	....	....
43	1.012	0.963	....	3.42	....	0.375	0.952	0.390	....	....
44	0.258	0.258	....	0.446	....	0.00174	1.000	0.00675	....	....
45	0.395	0.393	....	0.836	....	0.00783	0.995	0.0199	....	....
46	0.646	0.632	....	1.74	....	0.0712	0.979	0.112	....	....
47	0.822	0.819	....	2.50	....	0.0420	0.996	0.0513	....	....
48	0.950	0.940	....	3.11	....	0.0604	0.989	0.0642	....	....
49	1.038	1.026	....	3.56	....	0.0615	0.989	0.0599	....	....
50	1.036	0.993	....	3.55	....	0.266	0.959	0.268	....	....
51	0.929	0.795	....	3.00	....	1.53	0.856	1.925	....	....
52	0.823	0.715	....	2.50	....	1.065	0.869	1.49	....	....
53	1.036	0.944	....	3.55	....	0.728	0.911	0.771	....	....
(*c*) DATA ON DAM A SPILLWAY										
1	....	11.59	....	....	2.21	0.970	....	....	0.0545	0.650
2	....	11.59	....	....	0.990	2.34	....	....	0.0291	0.144
3	....	11.59	....	....	2.58	0.775	....	....	0.0623	0.931
4	....	11.59	....	....	3.31	0.532	....	....	0.0773	1.68
5	....	15.69	....	....	5.00	1.22	....	....	0.0571	0.735
6	....	15.74	....	....	3.84	1.73	....	....	0.0455	0.414
7	....	15.79	....	....	3.38	2.22	....	....	0.0401	0.286
8	....	15.84	....	....	1.52	5.90	....	....	0.0210	0.0563
9	....	19.64	....	....	7.32	1.66	....	....	0.0506	0.600
10	....	19.54	....	....	6.21	1.84	....	....	0.0461	0.490
11	....	19.52	....	....	4.83	2.76	....	....	0.0367	0.260
12	....	19.51	....	....	4.18	3.50	....	....	0.0320	0.179
13	....	19.49	....	....	2.12	9.68	....	....	0.0179	0.0359

[a] Values of q_a are in millionths, except Table (*c*) which is in cubic feet per second. [b] Feet of water.
[c] These values of H_o were not measured. They were computed with the assistance of the curve in Fig. 9.

and laboratory tests were undertaken to supply the needed information. Fig. 2 is a schematic diagram showing the dimensions which were measured in the various experimental setups.

The only previous experimental data which could be found were those referred to by J. W. Johnson,[2] Assoc. M. Am. Soc. C. E. His experiments were performed on a weir 1 ft long under heads varying from 0.171 to 0.305 ft (see Table 1(*a*)). In tests 1 to 76 of Table 1(*a*), air was allowed to recirculate beneath the nappe, giving rather uncertain values of q_a. In tests 83 to 106, recirculation was prevented by a sloping baffle beneath the nappe. Mr.

[2] "The Aeration of Sharp-Crested Weirs," by Joe W. Johnson, *Civil Engineering*, March, 1935, pp. 177–178.

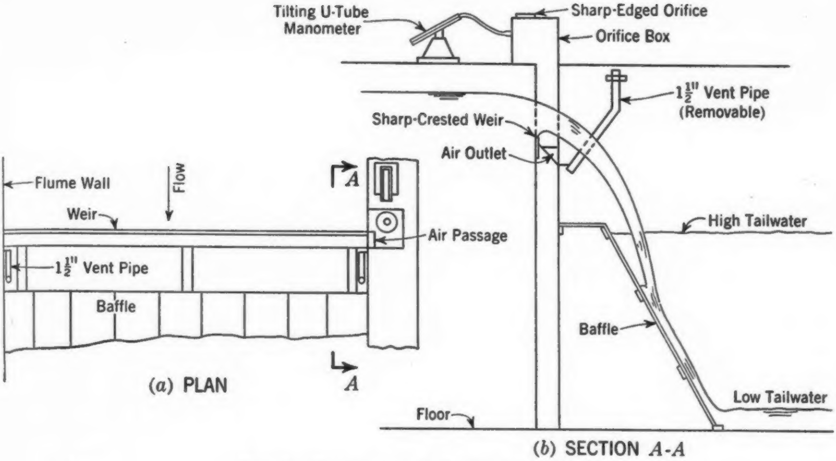


FIG. 3.—EXPERIMENTAL SETUP, 8-FT WEIR

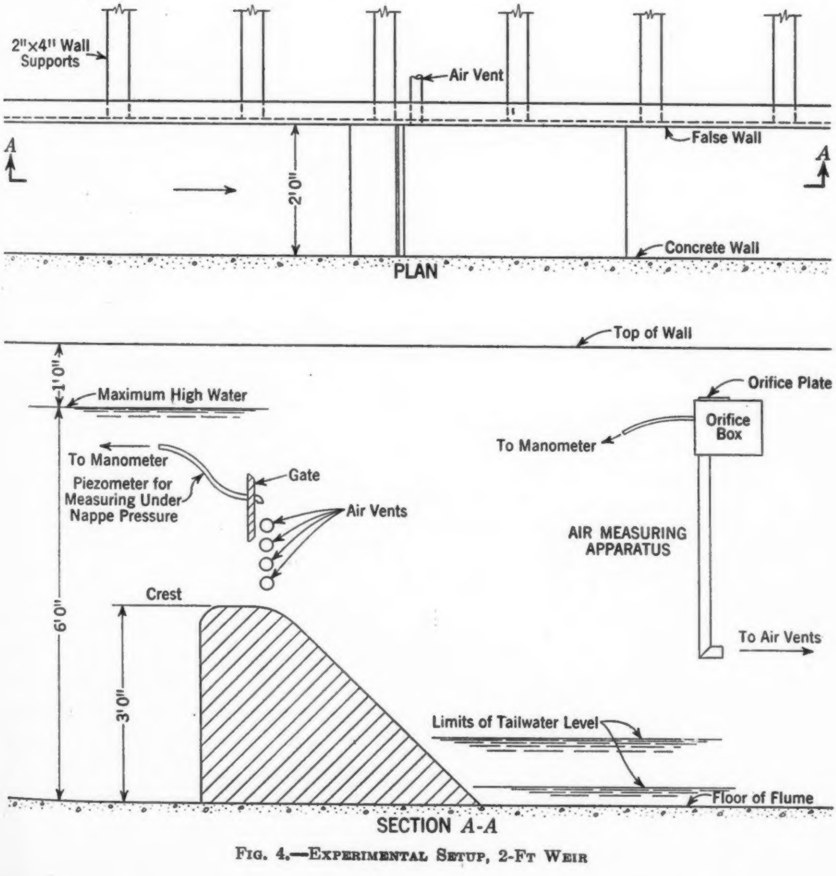


FIG. 4.—EXPERIMENTAL SETUP, 2-FT WEIR

Johnson obtained an expression for the quantity of air required per foot length of weir as

$$q_a = 0.0019\,(q_o)^{2.1} \qquad (1)$$

—for heads less than 0.4 ft; and

$$q_a = 0.0096\,(q_o)^{2.1} \qquad (2)$$

—for heads greater than 0.4 ft. In view of the variation of the coefficient at low heads, and the dimensional nature of the equations, it was thought advisable to conduct additional experiments before applying these equations to gates having crests 40 ft long and operating under heads of 20 ft.

Accordingly, the laboratory conducted a series of tests on a larger weir to extend the range of Mr. Johnson's data. An 8-ft suppressed weir available at the laboratory was fitted with an arrangement for measuring the air required. Fig. 3 illustrates the apparatus. Full aeration was obtained by two 1.5-in. pipes, placed one at each end of the weir, as shown. These pipes caused two openings about 2 in. wide the entire height of the nappe. When full aeration was not desired, the pipes were removed and the air admitted under the nappe was measured by orifices 0.25 in., 0.5 in., and 2 in. in diameter mounted on a box about 1 ft square. The air was conducted under the nappe through a duct placed in the slot supporting the weir bulkhead. The differential pressure on the orifice was measured with a tilting water manometer. The pressure under the nappe was not measured directly but was calculated as the pressure drop across the measuring orifice plus the pressure drop through the connecting duct. The correction was negligible for the smaller orifices and was not large for the 2-in. orifice. A sloping baffle was provided below the weir to eliminate the recirculation of air which Mr. Johnson

TABLE 2.—AERATION OF A TWO-FOOT, SHARP-CRESTED WEIR

(Various Gate Heights, Gate Opening y = 0)

Test No. (1)	H_o (ft) (3)	q_a (6)[a]	p (ft)[b] (7)	α (10)	β (11)
(a) GATE 0.5 FT HIGH					
85	1.143	42.1	0.0132	0.222	19.2
87	0.521	33.1	0.0079	0.494	32.6
90	0.512	7.58	0.0963	0.128	0.682
216	1.071	110	0.006	0.148	26.5
217	0.787	110	0.067	0.110	1.30
219	0.373	3.6	0.022	0.176	2.98
220	0.762	7.8	0.170	0.0760	3.41
221	0.769	11.5	0.223	0.0851	0.294
224	1.271	3.78	0.024	0.0515	2.73
225	0.243	1.90	0.006	0.272	11.0
(b) GATE 1.0 FT HIGH					
29	0.497	41.8	0.0102	0.544	26.5
30	0.704	40.6	0.0176	0.331	13.2
32	1.252	54.2	0.0215	0.204	11.9
34	0.392	23.9	0.0046	0.638	54.3
35	0.472	36.0	0.0095	0.542	27.0
39	1.246	144	0.0113	0.392	43.3
42	0.372	3.16	0.0171	0.176	3.82
43	0.469	3.14	0.0150	0.143	4.49
44	0.707	3.75	0.0232	0.0933	2.84
45	1.046	2.47	0.0105	0.0625	6.22
(c) GATE 2.0 FT HIGH					
143	1.051	15.2	0.0070	0.164	24.7
144	0.715	12.2	0.0067	0.226	24.2
150	0.963	36.8	0.0100	0.265	25.5
151	0.962	140	0.0093	0.525	54.3
152	0.738	130	0.0065	0.725	82.4
153	0.467	98.5	0.0037	1.15	145.0

[a] In millionths of a cubic foot per second. [b] Feet of water.

found in his experiments. The head on the weir varied from 0.258 ft to 1.097 ft. The tailwater level varied through the approximate range indicated in the figure. Discharges were measured by allowing the water to flow over the weir with the nappe completely ventilated. An accurate volumetric calibration for this condition of flow was available. The air requirement for this discharge was then measured with each of the orifices. A few runs were made with no air at all supplied under the nappe. Air and water temperatures were recorded. The data are given in Table 1(*b*).

In connection with the aeration requirements of dam D, additional tests were made to determine the effect of discharge beneath the gate as well as over it. A 2-ft weir, on which the head could be raised to 1.4 ft, was used for these tests. To prevent the back circulation of air drawn beneath the nappe, the lower face of the bulkhead was sloped so that the nappe fell on it. Three gates, 0.5 ft, 1.0 ft, and 2.0 ft high, were used with openings beneath them of 0, 0.1, 0.5, and 1.0 ft. The head over the crest varied from 0.095 ft to 1.409 ft. The air required was measured in the same way as that for the tests on the 8-ft weir. Discharges were measured by the calibrated 8-ft weir. Fig.

TABLE 3.—Aeration of a Two-Foot, Sharp-Crested Weir with Gate 0.5 Ft High

(Variable Openings y)

Test No. (1)	H_o (ft) (3)	H_u (ft) (3′)	q_a (6)[a]	p (ft)[b] (7)	α (10)	β (11)	ζ (12)
			(*a*) y = 0.086 Ft				
208	0.311	0.854	4.92	0.002	0.447	69.5	0.458
211	0.727	1.270	51	0.018	0.358	14.4	0.156
212	0.728	1.271	....	0.006	....	....	0.156
213	1.173	1.713	....	0.011	....	....	0.0890
214	1.409	1.952	70.5	0.028	0.194	9.75	0.0717
215	1.261	1.804	49.8	0.014	0.216	19.5	0.0816
			(*b*) y = 0.10 Ft				
66	0.446	0.996	11.6	0.202	0.152	0.336	0.334
67	0.218	0.768	4.22	0.0289	0.304	2.30	0.862
68	1.235	1.785	3.50	0.0208	0.0530	3.15	0.974
69	1.129	1.679	2.98	0.0151	0.0530	4.34	0.108
70	0.980	1.530	3.44	0.0200	0.0670	3.28	0.127
71	0.980	1.530	47.8	0.0164	0.262	15.7	0.127
73	0.795	1.345	51.6	0.0189	0.324	13.6	0.163
74	0.469	1.019	39.2	0.0109	0.549	23.6	0.314
77	0.637	1.187	120	0.0054	0.841	99.3	0.215
78	0.900	1.450	362	0.0727	0.542	6.70	0.141
			(*c*) y = 0.50 Ft				
52	0.912	1.662	62.0	0.0272	0.283	9.50	0.741
53	0.749	1.499	60.0	0.0225	0.356	11.8	0.943
54	0.416	1.166	41.0	0.0131	0.606	19.2	2.02
55	0.095	0.845	5.5	0.0003	2.51	794	15.7
56	0.189	0.939	....	0.0011	....	....	5.89
59	0.888	1.638	168	0.0153	0.553	32.1	0.765
60	1.019	1.769	224	0.0271	0.481	18.1	0.646
61	1.013	1.763	5.00	0.0384	0.0662	1.75	0.652
62	0.886	1.636	4.53	0.0343	0.0742	1.92	0.767
63	0.704	1.454	4.29	0.0319	0.0925	2.04	1.021
64	0.441	1.191	3.42	0.0198	0.149	3.31	1.845
65	0.160	0.910	2.27	0.0087	0.410	7.54	7.47
			(*d*) y = 0.577 Ft				
171	1.262	2.050	81.0	0.702	0.104	0.187	0.584
173	1.362	2.150	415	0.093	0.360	5.28	0.533
176	0.348	1.136	53.4	0.021	0.733	12.2	3.00
177	0.271	1.059	46.7	0.012	1.01	22.8	4.21
178	0.310	1.098	13.8	0.020	0.426	6.60	3.51
179	0.618	1.406	14.1	0.040	0.181	2.80	1.41
181	1.022	1.810	28.4	0.075	0.132	1.80	0.752
183	1.268	2.056	459	0.076	0.429	7.15	0.580
184	1.110	1.898	402	0.056	0.490	9.80	0.680
185	0.831	1.619	402	0.087	0.590	5.65	0.969
186	0.597	1.385	385	0.080	0.822	6.14	1.48
			(*e*) y = 0.934 Ft				
188	1.011	1.978	542	0.139	0.502	3.65	1.29
189	0.885	1.852	520	0.147	0.552	3.32	1.52
190	0.666	1.633	470	0.120	0.734	4.07	2.20
200	0.320	1.287	15	0.024	0.406	5.41	5.85
			(*f*) y = 1.00 Ft				
95	0.388	1.388	4.59	0.0357	0.169	1.84	4.87
96	0.197	1.197	1.86	0.0059	0.333	11.1	12.5
99	0.604	1.604	151	0.16	0.428	1.61	2.70
100	0.777	1.777	199	0.24	0.345	1.12	1.95
101	0.796	1.796	466	0.1109	0.625	4.49	1.89

[a] In millionths of a cubic foot per second. [b] Feet of water.

4 shows the general arrangement of the apparatus. Tables 2 to 5 give the data obtained.

In addition to the laboratory tests, it was possible to make observations of the actual air requirement of the dam A spillway. Fig. 5 shows the method of aerating this spillway. Each bay received air through two 16-in. pipes which terminated at their upper ends in a chamber at the down-stream face of the pier. Piezometers were provided so that the pressure under the nappe could be measured. The air supplied was measured by placing orifice plates over the entrances to the chambers at each end of the bay and sealing the pipes aerating the adjacent bays. A pressure connection on the orifice plate permitted the rate of air flow to be calculated from the pressure difference across the orifice. Fig. 6 shows the orifice plate and pressure connection. All pressures were measured by means of water or mercury manometers located on the spillway deck. The quantity of air required and the reduction of pressure beneath the nappe were varied for a given head on the gate crest by changing the orifice plates. The crest of the weir was 40 ft long, the heads varied from 11.59 ft to 19.04 ft, and the reduction of pressure beneath the nappe from 0.53 ft to 9.68 ft. Table 1(*c*) gives the data.

TABLE 4.—AERATION OF A TWO-FOOT, SHARP-CRESTED WEIR WITH GATE 1.0 FT HIGH

(Various Openings y)

Test No. (1)	H_o (ft) (3)	H_u (ft) (3′)	q_a (6)[a]	p (ft)[b] (7)	α (10)	β (11)	ζ (12)
(*a*) y = 0.1 FT							
15	1.135	2.185	172	0.0160	0.432	30.6	0.122
16	0.810	1.860	136	0.0128	0.569	36.0	0.187
17	0.386	1.436	86.2	0.0056	1.17	80.5	0.500
18	0.222	1.272	78.6	0.0036	2.16	133	1.08
19	0.219	1.269	3.44	0.0182	0.306	3.68	1.10
20	0.569	1.619	3.91	0.0272	0.114	2.38	0.297
21	0.821	1.871	3.37	0.0202	0.0790	3.21	0.184
22	1.063	2.113	3.31	0.0189	0.0615	3.46	0.133
23	1.239	2.289	4.24	0.0315	0.0525	2.06	0.110
24	1.242	2.292	63.7	0.0285	0.207	9.02	0.110
26	0.907	1.957	45.3	0.0154	0.280	16.5	0.162
27	0.547	1.597	53.8	0.0170	0.494	15.9	0.312
28	0.216	1.266	32.2	0.0079	1.17	31.9	1.12
(*b*) y = 0.5 FT							
4	0.113	1.363	1.5	0.0044	0.558	14.3	15.3
5	0.113	1.363	6.4	0.0040	1.19	33.5	15.3
11	0.941	2.192	255.4	0.0417	0.500	11.3	0.812
12	0.743	1.993	230	0.0323	0.640	14.8	1.10
13	0.508	1.758	168	0.0200	0.904	22.9	1.82
14	0.239	1.489	78.4	0.0168	1.37	19.5	5.23
46	0.172	1.422	27.3	0.0053	1.49	48.5	8.35
47	0.383	1.633	54.0	0.0208	0.671	12.3	2.69
48	0.565	1.815	73.3	0.0340	0.469	7.79	1.59
49	0.770	2.020	92.1	0.0608	0.335	4.25	1.05
50	1.038	2.288	83.5	0.0575	0.239	4.31	0.716
(*c*) y = 1.0 FT							
105	0.680	2.180	491	0.135	0.715	3.60	2.63
108	0.776	2.276	541	0.1736	0.619	2.77	2.20
109	0.767	2.267	186	0.28	0.326	0.894	2.24
110	0.601	2.101	154	0.1769	0.423	1.44	3.11
111	0.420	1.920	104	0.0739	0.620	3.52	5.09
112	0.196	1.696	35.8	0.0090	1.32	28.7	15.0
114	0.406	1.906	6.55	0.0619	0.168	1.10	5.34
115	0.625	2.125	11.4	0.2312	0.103	0.280	2.95
117	0.798	2.298	59.2	0.41	0.160	0.312	2.12
118	0.413	1.913	29.6	0.0921	0.318	1.42	5.22

[a] In millionths of a cubic foot per second. [b] Feet of water.

ANALYSIS OF TEST DATA

Air Required for Aeration.—The quantity of air supplied beneath the nappe in all of the experiments except Mr. Johnson's, both in the laboratory and in

the field, was measured by sharp-edged circular orifices and calculated by the hydraulic formula modified by the coefficient of Bean, Benesh, and Buckingham. For the case of discharge through an orifice from the free atmosphere, the equation reduces to

$$Q_a = (0.597 - 0.00844\,h) \times A \sqrt{2\,g\,h\,\frac{w_w}{w_a}} \quad \ldots (3)$$

in which: Q_a is the rate of air supply in cubic feet per second at atmospheric pressure; h = head on the orifice in feet of water; A = area; g = acceleration due to gravity; w_a = unit weight of air in pounds per cubic foot; w_w = unit weight of water; and B = length of weir crest, in feet.

The rate of air discharge in Mr. Johnson's experiments (Table 1(*a*)) was measured by means of a capillary tube.

The wide range of the variables involved in the experiments made it desirable to use a dimensionless form of representation. It was found that the relationship between pressure reduction, air required, and head over the top of the gate could be represented by plotting

TABLE 5.—AERATION OF A TWO-FOOT, SHARP-CRESTED WEIR WITH GATE 2.0 FT HIGH

(Various Openings y)

Test No. (1)	H_o (ft) (3)	H_u (ft) (3′)	q_a (6)[a]	p (ft)[b] (7)	α (10)	β (11)	ζ (12)
(*a*) y = 0.1 FT							
119	1.027	3.077	24.8	0.0296	0.155	5.38	1.68
120	0.606	2.656	23.3	0.0080	0.355	26.9	3.46
127	0.982	3.032	274	0.0130	0.662	50.0	1.78
(*b*) y = 0.5 FT							
131	0.614	2.864	320	0.0283	0.945	20.5	1.76
137	0.455	2.705	56.5	0.0040	0.873	99.3	2.68
155	0.614	2.864	24.0	0.059	0.215	2.24	1.76
156	0.627	2.877	23.2	0.056	0.209	2.34	1.71
157	0.469	2.719	14.4	0.027	0.267	4.64	2.57
158	0.330	2.580	9.7	0.010	0.397	13.1	4.24
159	0.330	2.580	10.2	0.011	0.397	11.9	4.24
162	0.332	2.582	11.0	0.013	0.395	10.1	4.19
163	0.423	2.673	38.8	0.0106	0.610	24.4	2.97
164	0.587	2.837	71.5	0.037	0.438	6.95	1.87
165	0.580	2.830	69.5	0.049	0.405	4.80	1.90
166	0.445	2.695	69.5	0.027	0.616	10.2	2.76
167	0.243	2.493	49.3	0.007	1.33	46.1	6.60
168	0.656	2.906	92.8	0.048	0.742	10.15	1.60
169	0.654	2.904	89.7	0.056	0.398	4.65	1.61
170	0.654	2.904	23.6	0.059	0.200	2.22	1.61

[a] In millionths of a cubic foot per second. [b] Feet of water.

$$\alpha = \frac{(q_a)^{0.5}}{g^{0.25}\,p^{0.25}\,H_o} \quad \ldots (4a)$$

against

$$\beta = \frac{(q_a)^{0.5}}{g^{0.25}\,p^{1.25}} \quad \ldots (4b)$$

in which: q_a = the rate of air supply per foot length of crest, $\frac{Q_a}{B}$; p = reduction of pressure beneath nappe, in feet of water; and H_o = head over the top of the gate, corresponding to the pressure reduction p. The length of crest was eliminated by assuming that each foot of length of overfalling nappe was independent of every other foot of length and that the total air required was proportional to the length. Fig. 7 is a plot of these variables for all tests in which discharge occurred only over the top of the gate. It will be seen that

the results obtained with the different experimental arrangements agree with each other within the limits of accuracy of the tests. It is particularly noticeable that Mr. Johnson's data on a 1-ft weir fall on both sides of the points obtained on the dam A spillway. The effect of the height of the overfalling

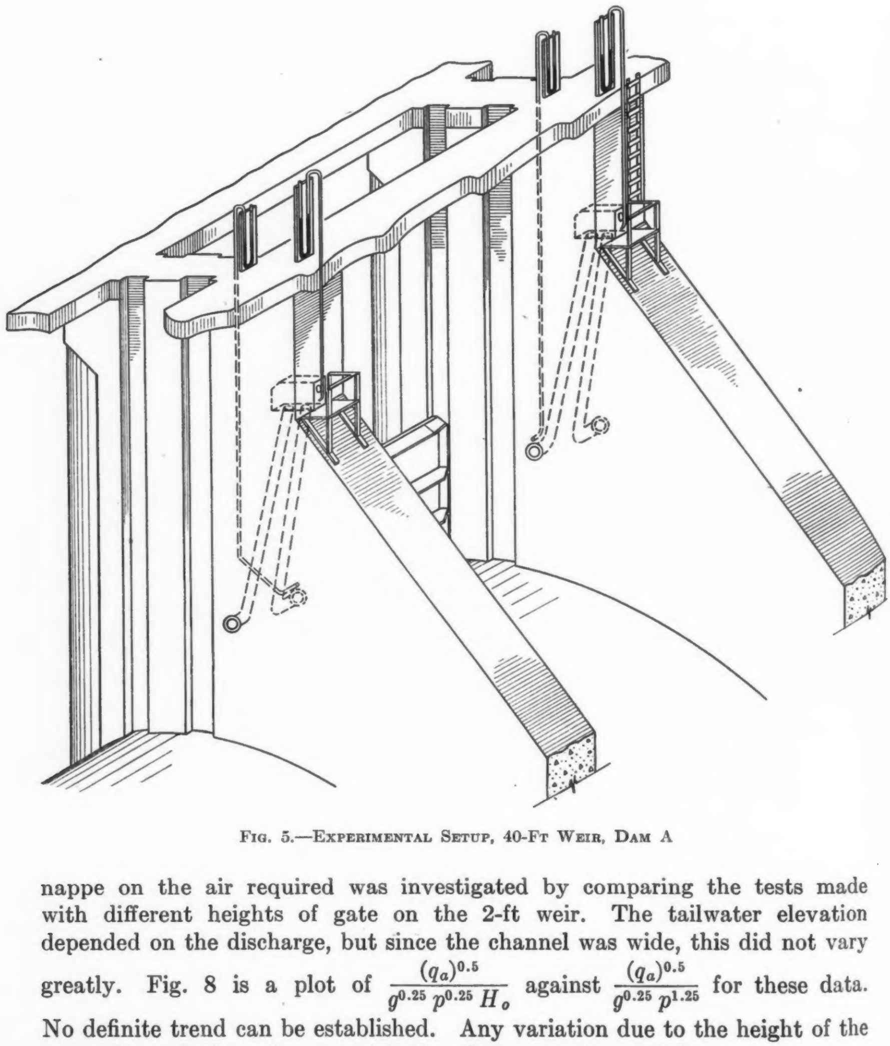


FIG. 5.—EXPERIMENTAL SETUP, 40-FT WEIR, DAM A

nappe on the air required was investigated by comparing the tests made with different heights of gate on the 2-ft weir. The tailwater elevation depended on the discharge, but since the channel was wide, this did not vary greatly. Fig. 8 is a plot of $\frac{(q_a)^{0.5}}{g^{0.25}\,p^{0.25}\,H_o}$ against $\frac{(q_a)^{0.5}}{g^{0.25}\,p^{1.25}}$ for these data. No definite trend can be established. Any variation due to the height of the nappe is masked by the uncertainties of the experimental data.

It was found that the results for the tests made with discharge beneath the gate could be represented satisfactorily by the dimensionless groups used for discharge over the gate, as long as the ratio of discharge beneath the gate to discharge over the gate remained constant. Since it was difficult to measure

the two discharges separately with the experimental setup used and since such separation would be practically impossible in field installations, the ratio of discharges was represented as follows: Let Q_o and Q_u be the discharge over and beneath the gate, respectively; then:

$$Q_u = C_u B y \sqrt{H_u} \quad \text{(5a)}$$

and

$$Q_o = C_o B (H_o)^{1.5} \quad \text{(5b)}$$

and the ratio of discharges is

$$\zeta' = \frac{Q_u}{Q_o} = \frac{C_u y \sqrt{H_u}}{C_o (H_o)^{1.5}} \quad \text{(6)}$$

If it may be assumed that C_u and C_o are constant, the ratio may be expressed as

$$\zeta = \frac{y \sqrt{H_u}}{(H_o)^{1.5}} \quad \text{(7)}$$

This form has the advantage of being composed of terms which are easily measured.

FIG. 6.—ORIFICE PLATE AND PRESSURE CONNECTION FOR MEASURING AIR SUPPLY AT DAM A

The ratio ζ was calculated for each test in which flow beneath the gate occurred (see Tables 3 to 5). All the data were then arranged in groups covering small ranges of ζ, as follows:

Range of ζ	Average ζ	No. of tests
0	0	93
0-0.4	0.17	20
0.4-1.2	0.81	24
1.2-2.3	1.79	24
2.3-3.3	2.81	10
3.3-4.5	4.0	6
4.5-5.5	5.1	5
>5.5	....	9

The data were plotted on separate sheets, using the groups $\frac{(q_a)^{0.5}}{g^{0.25}\, p^{0.25}\, H_o}$ and $\frac{(q_a)^{0.5}}{g^{0.25}\, p^{1.25}}$ as ordinates and abscissas, respectively, and the best straight line was drawn through each group. It was found that slight adjustments would make all the lines parallel with a slope of 0.45. Accordingly, the equa-

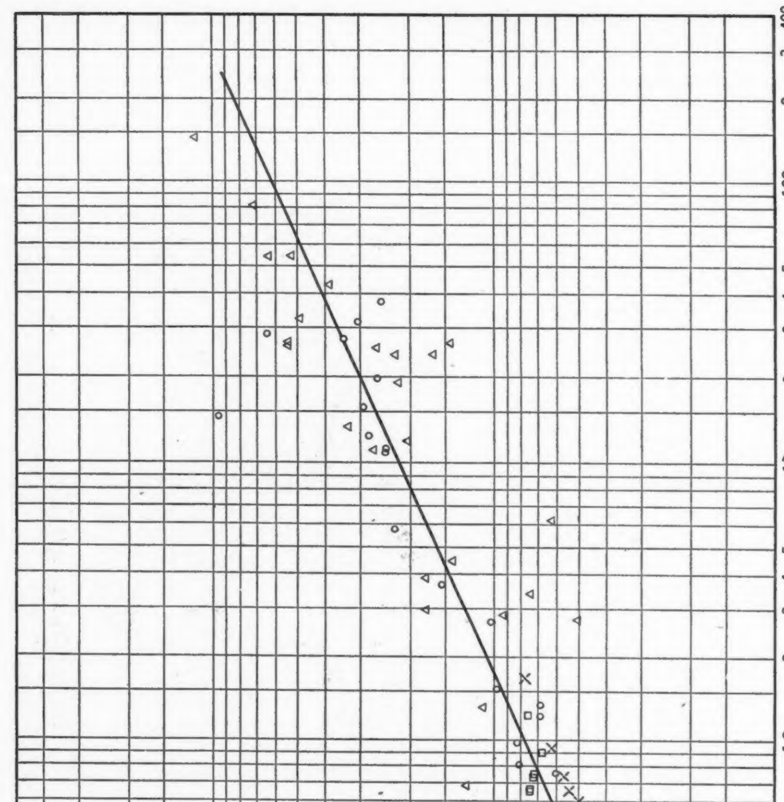


FIG. 7.—EFFECT OF WEIRS FOR $\zeta = 0$

tion of each line could be written

$$\frac{(q_a)^{0.5}}{g^{0.25}\,p^{0.25}\,H_o} = C\left[\frac{(q_a)^{0.5}}{g^{0.25}\,p^{1.25}}\right]^{0.45} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which C is a constant for each value of ζ. Fig. 9 shows the relation between C and ζ. The value of C increases rapidly for small values of ζ, but apparently

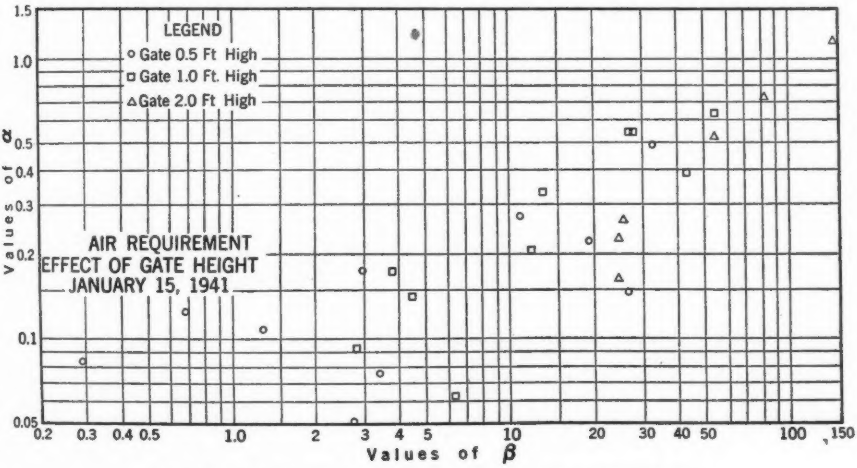


FIG. 8.—EFFECT OF GATE HEIGHT ON THE AERATION OF WEIRS

reaches a maximum value of 0.225 at $\zeta = 2.5$. Further increase in ζ does not increase C.

Eq. 8 may be solved for q_a:

$$q_a = \frac{(C\,H_o)^{3.64}\,g^{0.5}}{p^{1.14}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

Eq. 9 gives the quantity of air required per foot of length for a weir operating under specified conditions of gate opening, head, and pressure reduction beneath the nappe.

Effect of Pressure Reduction on Discharge.—The reduction of pressure beneath the nappe increases the effective head on the weir and causes the discharge for a given head to be greater than would be the case if the nappe were fully aerated. Similarly, for a given discharge, reduction in pressure decreases the measured head on the weir. The experimental data described in the preceding section provided a means of evaluating this effect. Fig. 10 is a plot of the ratios p/H_o and H_o/H'. A smooth curve has been drawn through the points.

The increase in discharge for a given head, caused by reduction of pressure beneath the nappe, can be calculated from these data. Suppose that the weir discharge at H is Q and at a slightly larger head H' is Q'. By reducing the pressure under the nappe, it would be possible to produce the discharge Q' with the head H. The increase of discharge at head H is then $Q' - Q$ and

FIG. 7.—EFFECT OF WEIRS FOR $\zeta = 0$

the ratio of this increase to the fully aerated discharge at head H is $\dfrac{Q' - Q}{Q}$. If the discharge coefficient of the weir is assumed to be constant, the ratio of increase in discharge may be written as $\dfrac{(H')^{1.5} - H^{1.5}}{H^{1.5}}$ or $\left(\dfrac{H'}{H}\right)^{1.5} - 1$. Fig. 10 gives the value of H_o/H' for any value of p/H_o. With this information it is a simple matter to calculate the increase in discharge. Fig. 10 also shows the percentage increase of discharge in terms of p/H_o. This curve is particularly useful in evaluating the effect of insufficient aeration on discharge where it is not possible to aerate the weir properly.

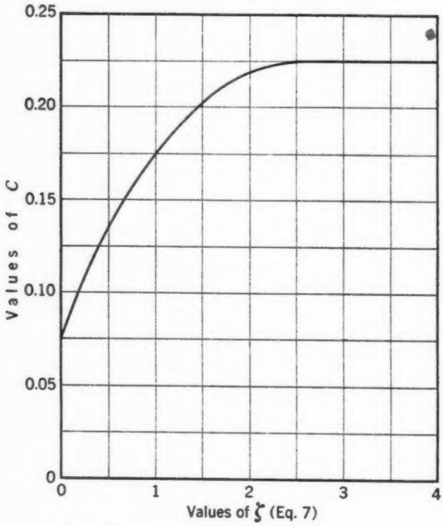


Fig. 9.—Relation Between C and ζ

Application of Results

Calculation of Aeration Vent Required.—Eq. 9 and Fig. 9 give sufficient information to determine the quantity of air required to maintain the pressure reduction beneath the nappe at any desired value. As an example, consider a spillway gate 40 ft long with water discharging over it under a head of 15 ft. Since there is no discharge beneath the gate, $\zeta = 0$. Fig. 9 shows that C for this condition is 0.077. It will be assumed that it is desired to limit the pressure reduction beneath the nappe to a maximum of 2 ft. Eq. 9 gives the air requirement for these conditions as $q_a = \dfrac{(0.077 \times 15)^{3.64}\, g^{0.5}}{2^{1.14}}$ = 4.35 cu ft per sec per ft of length of weir or 174 cu ft per sec for the 40-ft length of gate.

The vent diameter required to carry this amount of air may be computed by the use of the ordinary hydraulic equations. The effect on the area caused by neglecting compressibility is about 4% for a pressure reduction of 5 ft. In terms of diameter of vent, this difference is negligible. In calculating the flow of air, the effective head must be stated in terms of feet of air rather than feet of water. The value of p as given must be multiplied by the ratio of the unit weight of water to that of air. For air at 70° F, this ratio is approximately 830. In the example given, suppose that the most convenient method of venting is by the use of a pipe 30 ft long, with two right-angle bends and a sharp-cornered entrance. It is obvious that the total pressure drop through the vent pipe is equal to the pressure reduction beneath the nappe. A preliminary calculation of the pressure drop may be made with an assumed pipe diameter, say 1 ft, and an air temperature of 70°. For these conditions, the

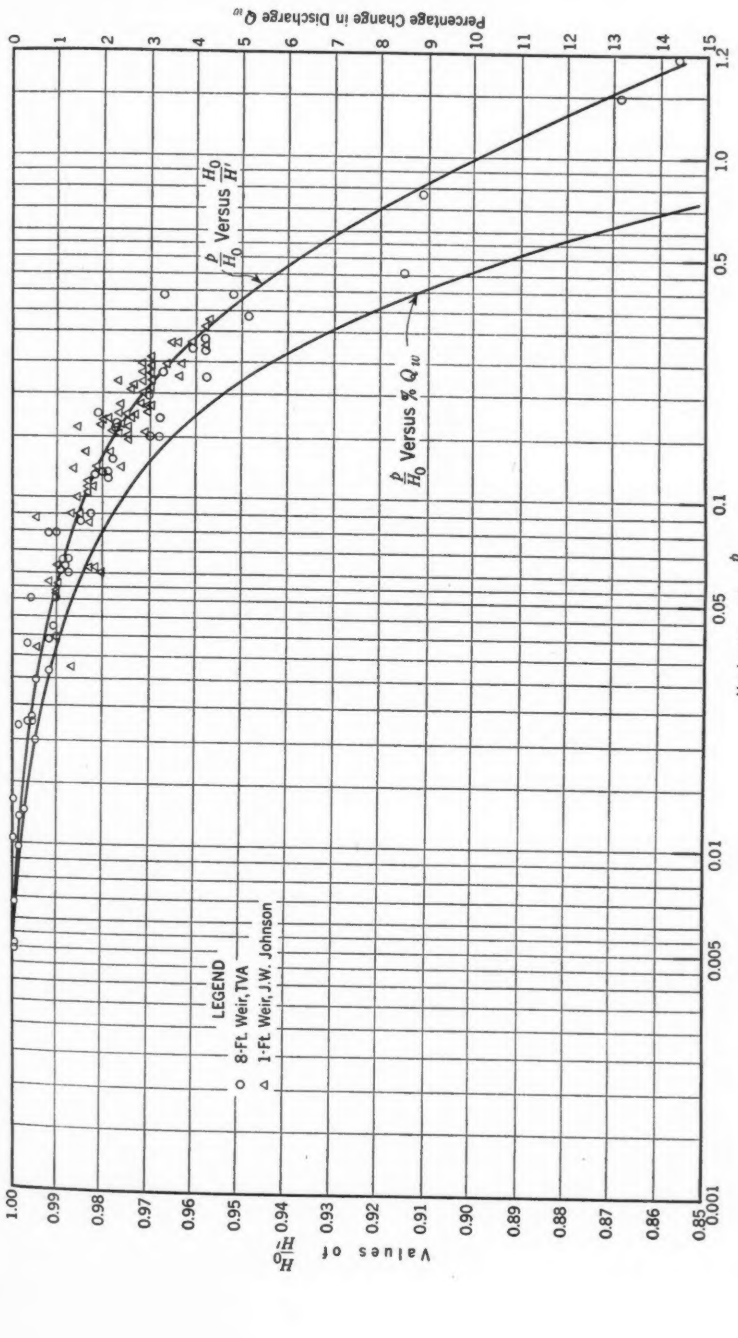


FIG. 10.—RELATION BETWEEN $\frac{p}{H_o}$ AND $\frac{H_o}{H'}$

pressure drop for the amount of air required is

$$p = \frac{w_a}{w_w}\left(C_e + \frac{f\,L}{D} + 2\,C_b + 1\right)\frac{V^2}{2\,g} \quad \text{(10)}$$

in which: $\frac{w_a}{w_w} = \frac{1}{830}$; $C_e = 0.5$; $f = 0.02$; $C_b = 0.5$; and $V = \frac{174}{0.785} = 222$. Substituting these values in Eq. 10,

$$p = \frac{1}{830}\left(0.5 + \frac{0.02 \times 30}{1} + 1 + 1\right)\frac{222^2}{2\,g} = 2.85 \text{ ft.}$$

Since the friction loss coefficient is relatively small, it may be considered independent of the diameter and the pressure drop is proportional to D^4. Then the required diameter is $D = \left(\frac{2.85}{2}\right)^{0.25} \times 1 = 1.09$ ft.

There is a relation between the vent diameter and the other variable factors which may be found by writing the equation for discharge through the vent

$$Q_a = q_a\,B = \frac{C'\,\pi\,D^2}{4}\sqrt{2\,g\,p\,\frac{w_w}{w_a}} \quad \text{(11)}$$

in which C' is a coefficient that combines the entrance, head, and friction losses. Solving for q_a and equating to Eq. 9, it is possible to solve for D:

$$D = C''\,\frac{(H_o)^{1.82}\,B^{0.50}}{p^{0.82}} \quad \text{(12)}$$

in which

$$C'' = \frac{2\,C^{1.82}}{\sqrt{C'\,\pi}\,\sqrt[4]{2\,\frac{w_w}{w_a}}} \quad \text{(13)}$$

Eq. 12 indicates that the diameter of vent required varies directly with the square root of the crest length, with the 1.82 power of the head on the crest, and inversely with the 0.82 power of the pressure reduction. (It will be noted in Eqs. 11 and 12 that since q_a has been given dimensions of cubic feet per second, B must be considered as dimensionless.) With this relationship established, it is necessary to go through the foregoing calculations only once for any particular weir or gate installation. The vent diameter required for any reasonable variations can be determined from Eq. 12. In the example, suppose the crest length were 50 ft, the head 20 ft, and the allowable reduction of pressure 3 ft. The diameter of vent required would then be $D = 1.09\left(\frac{20}{15}\right)^{1.82}\left(\frac{50}{40}\right)^{0.50}\left(\frac{2}{3}\right)^{0.82} = 1.47$ ft.

The procedure for the case in which water discharges beneath the gate as well as over it is very similar. The only difference is that the value of ζ is not zero. In the first example, suppose that the gate were 20 ft high and lifted 6 ft, the elevation of the headwater remaining the same. Then H_o would be 9 ft, y would be 6 ft, and $\zeta = \frac{y\,\sqrt{H_u}}{(H_o)^{1.5}} = \frac{6 \times \sqrt{32}}{9^{1.5}} = 1.26$. The corresponding value of C from Fig. 8 is 0.190. Eq. 9 gives the amount of air required as

$q_a = \frac{(0.190 \times 9)^{3.64} g^{0.5}}{2^{1.14}}$ = 18.1 cu ft per sec per ft of length or 724 cu ft per sec for the entire gate. If the same method of venting is used as in the first example, the vent diameter required is 2.17 ft. This condition should always be investigated where there is a possibility of its existence. The reduction of pressure which would be developed with the 1.09-ft vent can be found by rewriting Eq. 12 in terms of pressure

$$p = C''' \frac{H^{2.22} B^{0.61}}{D^{1.22}} \qquad (14)$$

from which $p = 2 \left(\frac{2.17}{1.09} \right)^{1.22}$ = 4.63 ft of water, which is more than double the allowable reduction of 2 ft.

Care should be taken to see that Eqs. 12 and 14 are used only in cases where the value of C is constant as this is a necessary condition of their derivation.

Calculation of Discharge for Incomplete Aeration.—Referring again to the first example, it is desired to know the actual discharge over the gate when the pressure reduction beneath the nappe is 2 ft of water. It may be known, from model tests, for example, that the discharge for a fully aerated nappe is 8,000 cu ft per sec. The ratio $\frac{p}{H_o} = \frac{2}{15} = 0.133$. Fig. 10 shows that the corresponding increase in discharge is 2%, or 160 cu ft per sec, making the total discharge 8,160 cu ft per sec for these conditions.

The data available are not sufficient to determine the increase in discharge when water is flowing both over and beneath the gate.

Conclusions

All of the available data on aeration of spillways have been collected and analyzed for the purpose of providing a workable criterion for the size of aeration vents required. The dimensionless groups selected do not reflect the effect of height of fall or angle of impact of overfalling nappe on spillway face or tailwater, although these factors would seem to be involved. Nevertheless, the agreement between small spillways 1 ft long at a head of 0.4 ft and a large spillway 40 ft long at a head of 19 ft is close enough for design purposes.

An interesting and useful by-product of the investigation was the effect of reduction of pressure beneath the nappe on the relation between head and discharge.

It is hoped that the discussion will bring out other data that will either support or disprove the results presented.

APPENDIX

Notation

The following letter symbols, used in this paper, conform essentially to American Standard Letter Symbols for Hydraulics, prepared by a Committee

of the American Standards Association, with Society representation, and approved by the Association in 1942:[3]

A = area, in square feet;
B = length of weir crest, in feet;
C = (see Eq. 8);
C_b = coefficient of bend loss;
C_e = coefficient of entrance loss;
C_o = discharge coefficient for flow over the gate;
C_u = discharge coefficient for flow under the gate;
C' = coefficient combining the entrance, head, and friction losses;
C'' = (see Eq. 13);
C''' = (see Eq. 14);
D = diameter of pipe, in feet;
f = coefficient of pipe friction in the Fanning formula;
g = acceleration of gravity, in feet per second^2;
H = head on the weir, in feet;
H_o = head over the top of the gate corresponding to a pressure reduction p, in feet;
H_u = head on the center of the opening below the gate, corresponding to a pressure reduction p, in feet;
H' = head over the top of the gate for full aeration, in feet;
h = head on the orifice, in feet of water;
L = length of pipe, in feet;
p = reduction of pressure beneath the nappe, in feet of water;
Q = discharge over a fully aerated weir at head H, in cubic feet per second;
Q_a = rate of air supply, in cubic feet per second, at atmospheric pressure;
Q_o = discharge over the top of the gate, in cubic feet per second;
Q_u = discharge beneath the gate, in cubic feet per second;
Q' = discharge over a fully aerated weir at the head H', in cubic feet per second;
q = discharge per unit width of weir, in cubic feet per second;
q_a = rate of air supply per foot length of crest, Q_a/B, in cubic feet per second;
q_o = discharge over the top of the gate per foot length of crest, Q_o/B, in cubic feet per second;
V = mean velocity, in feet per second;
w = weight per unit volume:
  w_a = unit weight of air, in pounds per cubic foot;
  w_w = unit weight of water, in pounds per cubic foot;
y = height of the opening below the gate, in feet;
α = a dimensionless group defined by Eq. 4*a*;
β = a dimensionless group defined by Eq. 4*b*;
ζ = a dimensionless ratio, $\dfrac{y\sqrt{H_u}}{(H_o)^{1.5}}$: ζ' = ratio of discharges Q_u/Q_o.

[3] ASA—Z10.2—1942.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PENDLETON LEVEE FAILURE

Discussion

BY RALPH B. PECK, JUN. AM. SOC. C. E.

RALPH B. PECK,[3] JUN. AM. SOC. C. E.[3a]—The results of a type of field test whose importance cannot be overestimated in the field of soil mechanics are presented in this paper. The ability to recognize the possibility for valuable observations in the circumstances surrounding the construction of the new levee, and to make the observations, is to be commended greatly.

A feature that would have greatly enhanced the value of the investigation is a more adequate foundation exploration. Inasmuch as the failure occurred along an essentially horizontal surface, the presence of a single lens or stratum of weak material, even if only a few inches thick, might have conditioned the entire failure. Such a foundation detail could easily have been responsible for the land-side failure of the structure when water-pressure measurements suggested the greater likelihood of a river-side failure. An adequate foundation exploration might reasonably include a large number of test borings from which continuous clay cores could be obtained without excessive disturbance. The entire vertical length of each core should be examined for the possible weak zones by means of laboratory tests, preferably unconfined compressive strength determinations on every sample. After sufficient tests had established a statistical relationship between the compressive strength and some simpler property, such as the natural water content, some reduction in compression testing might become possible, but only by a thorough investigation can such a foundation be properly studied. The important goal is the statistical picture of the variability of the strength of the deposit. In this connection, simple tests of many samples are of far greater value than the most elaborate investigation of a few samples.

One of the most interesting features of the results of the investigation is the fact that the total measured increase in hydrostatic pressure in the clay foundation layer was appreciably greater than the added weight of the embankment. The authors have taken this fact as an indication that the shearing

NOTE.—This paper by Kenneth E. Fields and William L. Wells, Assoc. Members, Am. Soc. C. E., appears on pp. 1763–1776 of this issue of *Proceedings*.

[3] Research Asst. Prof., Soil Mechanics, Univ. of Illinois, Urbana, Ill.

[3a] Received by the Secretary October 26, 1942.

resistance of the clay decreased as the embankment was constructed. Their computations even indicate that at certain critical points the effective angle of shearing resistance of the clay was reduced to 0°. This result is in agreement with the results of earth-pressure measurements against the temporary bracing of open cuts and against the temporary lining of tunnels constructed in Chicago (Ill.) clays for the subway. In these cases the field evidence for an inconsequential angle of shearing resistance is very strong. The shearing resistance in the field was in close agreement with the laboratory results on simple, unconfined compression tests. Furthermore, the mobilization of that part of the shearing resistance which was effective, and the simultaneous loss of frictional resistance, occurred at a relatively small strain. This amount of strain appeared to be so small in the case of the Chicago open cuts that it may be considered to be the minimum strain inevitably associated with the development of the shearing resistance. It is possible that the term "remolding" is too extreme to describe this phenomenon. The best so-called "undisturbed" samples have probably experienced greater strains in the sampling process.

In spite of the aforementioned field evidence, it has not been possible in the laboratory to subject samples to shear tests which appear to be analogous to the conditions in the field and to duplicate the result of a small angle of shearing resistance. The consolidated, quick type of test produces an apparent angle of shearing resistance of approximately 17° for almost any clay. Whether the small disturbance caused by the sampling is sufficient to overshadow subsequent laboratory procedure is unknown. The fact remains that, so far, a discrepancy exists between the results of field and laboratory that constitutes an important gap in current knowledge. Only the unconfined compression

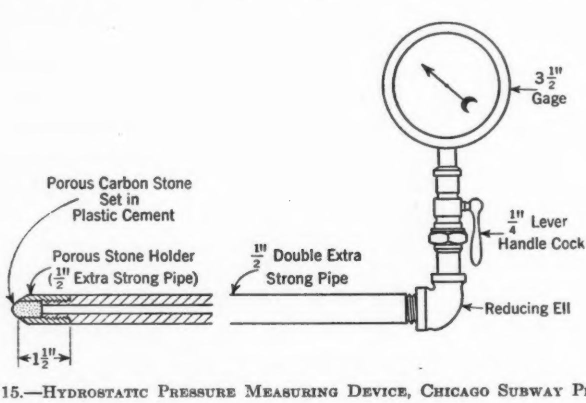


FIG. 15.—HYDROSTATIC PRESSURE MEASURING DEVICE, CHICAGO SUBWAY PROJECT

test, free from lateral pressure or surcharge, seems to give values of shearing resistance in accord with the field measurements.

Measurements of earth pressures and of accompanying hydrostatic pressures in regions where large shearing stresses are anticipated may constitute an important source of future knowledge. The water-pressure measurements

described in this paper appear to have been satisfactory. Other types of installations may be desirable for attacking the same problem under different circumstances. For measuring water pressures in Chicago clays, for example, the device shown in Fig. 15 was developed. It consists essentially of an ordinary Bourdon gage, a length of pipe, and a porous tip, all assembled and completely filled with water before installation. Because of the low permeability of clays, it is extremely important that no leakage should occur in the measuring system. The Chicago devices possess the advantage that the porous tip can be replaced in the laboratory by a pipe cap and the entire assembly subjected to a large hydrostatic pressure for a sufficient length of time to detect any leaks. The porous stone is then substituted for the pipe cap. The assembly does not require the drilling of a hole in advance. It is

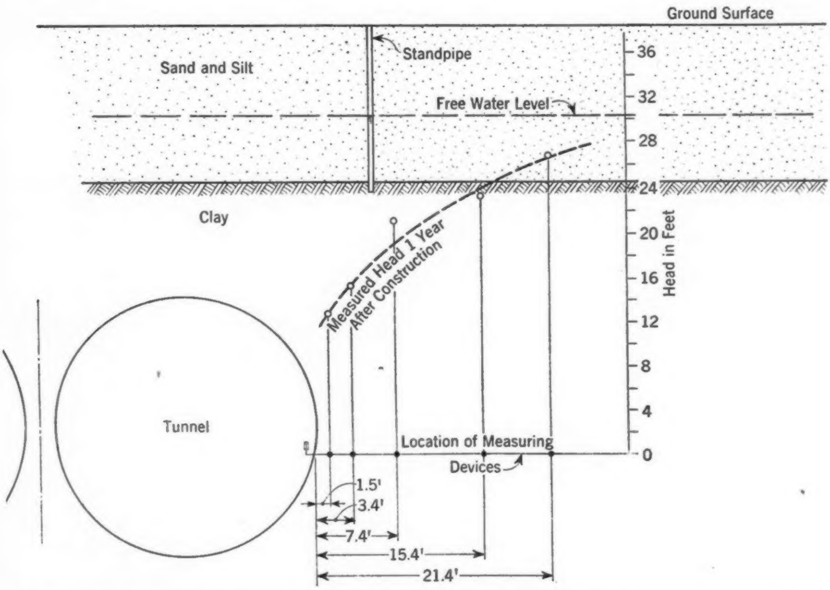


FIG. 16.—WATER PRESSURE MEASUREMENTS; CHICAGO SUBWAY EXPERIMENTAL SECTION S-3

merely jacked into the clay. It may be used in any position as long as the pressure is positive at the elevation of the pressure gage, a restriction which also applies to the Pendleton devices. In the Chicago clays, which have a permeability of approximately $5 \times (10)^{-8}$ cm per sec, the time necessary to reach equilibrium was between 12 hr and 24 hr.

The Chicago devices have proved satisfactory in measuring hydrostatic pressures around the subway tunnels. An example of the results is shown in Fig. 16, where a decrease in water pressure around the tunnel tubes, due to their action as drains, was measured, in spite of the fact that the free water surface remained constant over the tunnels because of the presence of a pervious silt layer in which it lay. Similar installations could provide valuable infor-

mation concerning the excess water pressure along the theoretical failure surfaces adjacent to open cuts. Although the water devices were not developed to a satisfactory point in time to perform such experiments on the initial system of Chicago subways, it is hoped that the success of the water-pressure measurements described in this paper, and those elsewhere, will encourage additional measurements to obtain evidence on this extremely important aspect of the shearing resistance of natural clay soil.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ORGANIZING AND FINANCING SEWAGE TREATMENT PROJECTS

Discussion

BY MESSRS. MILTON P. ADAMS, AND HAL F. SMITH

MILTON P. ADAMS,[3] M. AM. SOC. C. E.[3a]—In the "Synopsis" of this paper, Mr. Greeley limits the scope to "organizations established for the single purpose of sewage treatment and having this limited objective." The writer, in turn, since there are no such organizations in Michigan, must limit his discussion to a listing of the causes underlying the failure to achieve true metropolitan district financing and administration of the sewage treatment function in the area around Detroit. He will also mention briefly some of the problems that have been encountered elsewhere in the State of Michigan in order that the larger community may provide sewage collection and treatment to its smaller neighbor.

An amendment (Art. VIII, Sec. 31) to the State Constitution of Michigan, together with Act 312, Public Acts of 1929, as amended by Act 56, Public Acts of 1935, provides the legal foundation "for the incorporation of any two or more cities, villages or townships or parts of same" into metropolitan districts. Sewage treatment is but one of several objectives attainable through such organization. An intermediate but necessary step to the formation of such a district is the appointment and organization of a charter commission. Rules for selecting representatives on such commission are provided by the enabling statute. Instead of representation being based on outstanding local and non-partisan qualification for such service, as under the Illinois Sanitary District law, unfortunately, in Michigan, one representative must come from each unit of government irrespective of size, whereas large cities are entitled to only one representative for each 200,000 population, over and above the first 200,000 of population, for which four representatives are allowed.

The charter subsequently must receive a favorable vote from each governmental unit. It is thereafter the magna charta of future administration, financing, and operation of the function sought.

NOTE.—This paper by Samuel A. Greeley, M. Am. Soc. C. E., appears on pp. 1727–1742 of this issue of *Proceedings*.

[3] Executive Secy.-Engr., Michigan Stream Control Comm., Lansing, Mich.

[3a] Received by Secretary November 27, 1942.

Before sewage treatment was actually under way in Detroit as a city project, Wayne County administration of this service was seriously proposed and reported upon, both as a county drain commissioner's function and as that of the board of supervisors.

A regional committee on sewage collection and treatment, headed by the Hon. William A. Comstock, Governor of Michigan (1933–1934), was organized in 1933. Its objective was the solution of the problems not only of Wayne County's municipalities along the Detroit River but those of southern Oakland and Macomb counties as well. The latter counties border Wayne County on the north.

The following factors appear to account for the failure to achieve a separate and independent regional organization for sewage treatment in the metropolitan area:

1. The predominating size and political relation of Detroit with reference to some forty surrounding or neighboring municipalities of Wayne, Oakland, and Macomb counties;

2. The statute provision, previously mentioned, in which one city with more than 85% of the tributary population and taxable property in the area would have been unable to exercise a controlling voice in the drawing of a charter necessary to the creation of such a district;

3. The situation created in 1935 when, with two applications before it—one by the City of Detroit, and one for Wayne County including Detroit—the PWA chose to deal with Detroit, leaving, for a subsequently submitted Wayne County application, the solution of the problems of the remaining municipalities not served by the Detroit project; and

4. The curb on metropolitan district powers of taxation and bonding inherent in the so-called 15-mill amendment to the State Constitution which became effective in 1933.

The nearest approach to regional administration of sewage treatment in Michigan is now operative under the provisions of Act 342, Public Acts of 1939, as amended by Act 353, Public Acts of 1941, in behalf of those communities in Wayne County not served by the Detroit project. During the spring and summer of 1942, under the stress of the war emergency and further conditioned upon favorable federal action as to the financing of certain necessary capital improvements, Wayne County has become the agent to serve the Grosse Pointe communities along Lake St. Clair as well as certain southern Macomb County municipalities. The treatment of all sewage so collected is by the City of Detroit acting under contract with Wayne County. A Sewage Disposal Committee of the Wayne County Board of Supervisors administers this function through its Board of County Road Commissioners.

Since October, 1939, Wayne County sewage treatment service has been available to its customer municipalities and townships at a uniform base rate independent of location. Until July 1, 1941, the basis of charge was \$26 per million gallons of sewage, as measured by water passing the user's water meter. The current rate is \$43 per million gallons, less discount, intended to be sufficient compensation to the customer governmental unit for collection of local

service charges. The county's contract, therefore, is with the municipal subdivision served and not with the individual user.

The Detroit sewage treatment plant is now operated by the Detroit Water Board. The City Department of Public Works, on the other hand, is responsible for sewer maintenance and controls both city and out-of-city connections to the sewer system.

Only within the past year (1941) has the Common Council of Detroit adopted a so-called metropolitan rate for out-of-city and out-of-county sewage treatment service. To this rate, however, the City Department of Public Works seeks reimbursement as well for flowage or trunkage rights through that portion of its system not provided with PWA assistance. Distantly located municipalities find themselves at a financial disadvantage, therefore, as compared with those located more closely to the city's single sewage treatment plant.

During the summer of 1942, under authority of the county sewage disposal law, the City of Detroit and County of Oakland have reached an understanding whereby the sewage of several southern Oakland County municipalities will be received and treated by the City of Detroit. At the November 3, 1942, election, the municipalities of Ferndale, Royal Oak, Hazel Park, Huntington Woods, Berkley, Pleasant Ridge, and Clawson voted to so amend their respective charters as to enter contracts with the County of Oakland for treatment of their sewage on the county plan.

No privately owned sewage treatment utilities exist in Michigan, although Act 320, P. A. 1927, as amended, provides for their creation and control. Attempts were made to organize sanitary districts at Traverse City, Owosso, and Monroe during 1928–1930 without success. Failure of these efforts is charged to the 8% to 10% return then asked on the necessary capital outlay plus operating charges. Perhaps, reluctance on the part of Michigan communities to encourage additional types of privately owned, public utilities was also a factor.

Among the problems encountered under contract relation between governmental units for sewage treatment service, where well-conducted sanitary district organizations are lacking, are the following:

1. Reluctance to enter into the initial agreement because of uncertainty as to future year-to-year costs;

2. The governmental unit desiring accommodation may be asked to pay any amount in excess of the pro-rata service cost up to such amount as the "traffic will bear" at the will of the party providing the service;

3. The shifting legal responsibility for pollution where shut off or by-passing takes place, in an effort to enforce payment or obtain concessions as to charges; and

4. The Grand Rapids-East Grand Rapids problem of continuity of service to the latter municipality and compensation to the former was assured by a county circuit court hearing and signing by the judge of a contract mutually agreed upon in advance by representatives of the two cities. This contract has a "reopening clause" which has never been invoked since the original agree-

ment was made early in the 1930's. The friendly suit or court hearing attested to by the county judge apparently has overcome the charter limitations of the municipalities which prevents their so contracting as to bind future legislative bodies of the two cities.

HAL F. SMITH,[4] ESQ.[4a]—A scholarly presentation of data concerning the development of organization and finance methods for sewage disposal projects is offered by Mr. Greeley. The paper includes a carefully considered description of the trend that the course of development is following. The writer's opinions, which are based entirely on personal experience in the field, are in accord with the author's findings.

The City of Detroit, Mich., made use of its regular municipal organization for construction and operation of its sewage disposal system, and assigned to its Department of Public Works, which includes the office of the city engineer, the task of constructing the system, which consists of intercepting sewers, sewage treatment plant, effluent conduit to the river, and its appurtenances. After completion, the operation of the system, including billing and collecting for service rendered, was transferred to the Department of Water Supply. The regular municipal organization was used because Detroit believes that, in general, economies in operation of municipal departments or projects lie in the path of consolidation of existing agencies rather than in the creation of new agencies.

The City had little choice in the matter of financing its sewage disposal project. It had already reached its bonding limit and did not have sufficient cash resources to finance the project; so it decided to use the city funds that were available, accept the PWA grant, and issue revenue bonds to finance the remainder. The revenue bonds were issued by authority of a city ordinance, supported by a state enabling act, which also provided for raising funds to meet the system's debt charges and operating expense by imposing a sewage disposal charge upon the users of the system. It was decided to base the sewage disposal charge on metered water consumption because it appeared that water consumption was the most satisfactory and equitable measure of the service rendered by the sewage disposal system.

Its rate structure must be considered a two-part rate, wherein the part based on usage consists of a single step. It is a two-part rate because a substantial part of the capital costs of the project were paid from tax revenues; consequently, the "usage" part of the rate is required to produce only enough revenue to meet debt charges of the part that was financed through sale of revenue bonds and to meet operating expenses and reserve fund requirements.

A single-step for the "usage" part of the rate was adopted for the reason that the entire charge is for treatment and disposal of sewage that is picked up by the interceptors; consequently, the cost of treating and disposing of any given unit of sewage is identical to the cost of treating and disposing of any other unit, regardless of whether the unit was contributed by one large contributor or many small ones.

[4] Senior Administrative Asst., Detroit Dept. of Water Supply, Detroit, Mich.

[4a] Received by the Secretary December 2, 1942.

Consideration was then given to the problem of whether the charge should be billed on a separate bill form, or as a separate item on the water bill, or included in the water bill. For reasons of economy it was decided to include the sewage disposal charge in the water bill and to make a clear explanation of this combined billing on the bill form. This choice of procedure has made it possible for the department to take over the job of billing, collecting, and accounting for sewage disposal charges without any additional expense. In general, customers have understood the additional charge for sewage disposal and the combined form of billing, with the result that there has been no appreciable effect on customer relations. However, it did necessitate the preparation of a plan that could be depended upon to effect a complete and accurate segregation of water and sewage disposal receipts.

The question was then considered as to who should be held responsible for sewage disposal charges. Inasmuch as water charges in Detroit are a lien against the property served, the sewage disposal ordinance was drawn to make the sewage disposal charge also a lien and enforcible in exactly the same manner as is the lien for the water charge.

The next question considered was that of determining a satisfactory means of forcing payment of sewage disposal charges. Although the right of lien is a formidable weapon for use against delinquent customers, it is a most cumbersome and costly one because of the expense of foreclosure, which usually exceeds, to a considerable extent, the ordinary sewage disposal or water charge. Its principal value lies in its ability to fix responsibility and depends largely upon some other agency to force actual collection. The perfect combination is the right of lien to establish liability and the right to discontinue the sewer service or water service to the property for nonpayment of the sewage disposal service. The Detroit ordinance was drawn to provide for all of these measures.

Detroit makes a charge for sewage disposal service to all properties that are connected to the system, on the basis of water consumed, making adjustments under the following rule adopted October 20, 1941:

> The Sewage Disposal rate shall be applied uniformly to all water consumers, whose property is connected to the Sewage Disposal System, on the basis of the quantity of water used thereon or therein, regardless of the purpose for which the water is used, or the character or concentration of the sewage delivered from the property to the Sewage Disposal System, except that an additional charge may be made where the character of the sewage is such that it imposes an unreasonable burden upon the system, as provided for in the city ordinance; provided, however, that upon proper showing, exemption of the sewage disposal charge will be made on water delivered through a metered line where the entire amount of water delivered through said line is used for such a purpose, and in such a manner as to establish beyond reasonable doubt the fact that the water so taken does not enter the Sewage Disposal System.

Of particular interest is the fact that, since the sewage disposal charge has been in effect, less than thirty requests have been received for adjustment of the sewage disposal charge on the grounds that all or part of their water consumption did not reach the sewer. In general, the requests for adjustment were from railroads and large industrial plants.

An ordinance was passed by the Common Council requiring users of private wells, or any other private sources of water supply, to meter or measure such supply in a manner satisfactory to the Department of Water Supply, as a means of determining the proper charge for sewage disposal service. The regular sewage disposal charge is applied to water supplies from private sources.

Contrary to a rather widespread opinion that sewage disposal charges are difficult to collect and, that if added to water bills, would tend to slow up water collections, experience in Detroit indicates that inclusion of the sewage disposal charge in the water bill has had little if any effect on collections. As a matter of fact, the department's customer delinquent balance in October, 1941, hit a new low for the preceding thirteen-year period, in spite of the fact that the delinquent balance since June 19, 1940, carries sewage disposal charges as well as water service charges.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ALLOCATION OF THE TENNESSEE VALLEY AUTHORITY PROJECTS

Discussion

BY THEODORE B. PARKER, M. AM. SOC. C. E.

THEODORE B. PARKER,[14] M. AM. SOC. C. E.[14a]—The discussions have elicited much additional information which undoubtedly will be of value to those interested in the subject. It is particularly gratifying to note that the discussers seem to be in substantial agreement with the methods used by the Tennessee Valley Authority.

Colonel Elliott agrees generally with the methods used in determining the costs to be allocated to the various purposes of a multiple-purpose project, but questions the use of a cost allocation, in particular with reference to the so-called power "yardstick." If it were conceded, for the purpose of the argument, that the costs of private and public enterprises were different, or, for that matter, that the costs of construction or types of structures required in different parts of the country were different, it would seem to the writer that adjustments for these differences could be made readily. Without a cost allocation to which such adjustments could be applied, there would be no base from which to begin in making any comparison. It should also be kept in mind that the capital cost of the generating stations is only a comparatively small part of the cost of power and that transmission, distribution, and marketing costs are not subject to allocation. It follows that even if the cost of generation for a public enterprise were 70% of the cost of generation of a private enterprise, the actual costs of power at market would be more nearly equal.

Mr. Chandler presents, in some detail, interesting variations of the alternative-justifiable-expenditure method which will greatly assist the uninitiated in a better understanding of the problem. However, it appears to the writer that, in the use of the so-called "three-circle" method, simplicity of the alter-

NOTE.—This paper by Theodore B. Parker, M. Am. Soc. C. E., was published in December, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1942, by Malcolm Elliott, M. Am. Soc. C. E.; March, 1942, by E. L. Chandler, M. Am. Soc. C. E.; May, 1942, by Eugene L. Grant, M. Am. Soc. C. E.; June, 1942, by Messrs. Sherman M. Woodward, and Albert R. Arledge; and October, 1942, by C. W. Watchorn, Esq., and F. A. Allner, M. Am. Soc. C. E.

[14] Chf. Engr., TVA, Knoxville, Tenn.

[14a] Received by the Secretary November 19, 1942.

native-justifiable-expenditure method is sacrificed without much, if any, gain in accuracy since a considerably larger number of estimates of cost are required.

Professor Grant raises an interesting and pertinent point by requesting more information on the effect of different plans of operation on the cost allocations. In accordance with the TVA Act, flood control and navigation are the primary purposes, and power is the by-product of the development of the Tennessee River and its tributaries. The Act is clear and specific, the various developments are planned and operated accordingly, and in normal times, therefore, there cannot be any choice as to plan of operation; the cost allocations are made to conform with this plan. However, the Authority has in operation (among others) the Cherokee Dam on the Holston River and the Nottely and Chatuge developments in the upper Hiwassee drainage basin, which, by special appropriation of the Congress, were authorized to alleviate a threatened power shortage in the region. During the hearings before the Congressional committees on the bill authorizing the construction of these projects, it was fully explained that these dams were to be operated temporarily for the primary purpose of maximum power production, and that at the conclusion of the war the operation of the dams would revert to the normal multiple-purpose operating plan.

To conform to the realities of this situation, the writer is recommending that the Authority submit two cost allocations for approval by the President of the United States—one to apply for the duration of the war and charging the entire cost of the additional developments to power, the other to go into effect after the Congress has declared the war emergency to be ended and giving effect to the planned multiple-purpose operation of the projects.

It is interesting to note that the methods used are sufficiently flexible to allow for this difference in operation as well as sufficiently accurate so that the difference in operation does not disappear in the process of allocation.

Mr. Woodward's discussion is a worth-while addition to the conception of allocations, and offers an alternative allocation principally involving the cost of Hiwassee Dam and affecting only the allocated costs of navigation and flood control.

Mr. Arledge summarizes clearly and concisely the various steps taken by the Congress with regard to the financing of the Boulder Canyon project. This discussion contains much valuable information which the individual engineer could only find after considerable research, if at all. The remarks of Mr. Watchorn and the late Mr. Allner do not appear to be particularly pertinent to the solution of the problem of cost allocation of the TVA projects as directed by the Congress. However, their discussion presents a thoughtful opinion on the general subject of allocations.

In conclusion, the writer wishes to express his sincere appreciation to all the discussers.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DEVELOPMENT OF TRANSPORTATION IN THE UNITED STATES

Discussion

BY J. E. TEAL, M. AM. SOC. C. E.

J. E. TEAL,[23] M. AM. SOC. C. E.[23a]—Since the first presentation of this paper in December, 1941, transportation conditions in the United States have changed materially. This change was first manifest in the summer of 1939, and became more pronounced after September 1, 1939, when the war in Europe was started, which resulted in industries in the United States receiving large orders for materials and supplies. When a major change occurs in industrial activity, all transportation agencies immediately feel the effect. This increased industrial activity continued to December 7, 1941, when the United States declared war.

The discussions have been enlightening, as they have revealed points omitted or overlooked by the writer and have expanded others. Because of the great change in transportation conditions, caused by the war effort, it appears to the writer that the discussion might well be terminated by emphasizing the relative importance of the various transportation agencies in connection with the war economy and let the record speak for itself.

The desire of transportation agencies is to assist in winning the war at an early date. The war economy of the United States depends largely on available means of transport. The ability of the Nation to transport men and materials overseas is of vital concern. However, the overseas shipments are greatly exceeded in volume by those within the United States, because here it is necessary to transport vast numbers of troops and military equipment from one point to another, in the process of training, and also to transport the materials required in the construction of military establishments, in the building of ships, and in the manufacture of ammunition, guns, planes, and tanks. All of this transportation for the war effort is given first consideration, of course,

NOTE.—This paper by J. E. Teal, M. Am. Soc. C. E., was published in December, 1941, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: February, 1942, by Messrs. Fred Lavis, and William J. Wilgus; March, 1942, by Joseph B. Eastman, Esq.; April, 1942, by Messrs. J. L. Campbell, W. W. Crosby, and W. B. Irwin; and May, 1942, by F. R. Schanck, M. Am. Soc. C. E.

[23] Transportation Engr., C. & O. Ry., Richmond, Va.

[23a] Received by the Secretary November 23, 1942.

and the remainder of the transportation is used for the benefit of civilian activities. The civilian population now (December, 1942) finds traveling more difficult because of the greatly increased demands on transportation agencies.

At the beginning of 1942 the Nation's transportation plant and facilities consisted of approximately 233,000 miles of railway first main track; 1,960,000 freight-train cars, of which 1,685,000 were railroad owned; 38,000 passenger-train cars; 45,000 locomotives; 1,469,000 miles of improved highway, on which were operated 5,000,000 highway trucks, 140,000 buses, and 29,000,000 automobiles; inland waterways, excluding the Great Lakes, which were handling 4% of the Nation's bulk freight; and 105,000 miles of pipe lines, which were moving 10% or more. Vessels that were operated on the Great Lakes and along the coast provided transportation at low cost for large quantities of coal, oil, and lumber. Airplane transportation was increasing at a rapid rate to handle the ever-increasing passenger, mail, and express load.

As stated by Ralph Budd,[24] Hon. M. Am. Soc. C. E.:

> "These various forms of transportation took on the traffic load incident to war abroad and preparedness at home, as it came along, without any considerable strain simply by utilizing a larger proportion of their carrying capacity than had been needed during the preceding years. Thus for the first time they realized the benefits from the extensive improvements that had been made."

Since the outbreak of World War II, on September 1, 1939, the railroads have handled a continuously increasing traffic load, as shown by Table 10.

TABLE 10.—INCREASE IN FREIGHT AND PASSENGER TRAFFIC ON UNITED STATES RAILROADS

Year	Car Loadings			Revenue Ton-Miles			Revenue Passenger-Miles		
	Thousands (1)	Index No. (2)	Increase (%)[a] (3)	Millions (4)	Index No. (5)	Increase (%)[a] (6)	Millions (7)	Index No. (8)	Increase (%)[a] (9)
1939	33,911	100.0	....	333,438	100.0	....	22,651	100.0	....
1940	36,358	107.2	7.2	373,253	111.9	11.9	23,762	104.9	4.9
1941	42,285	124.7	16.3	475,072	142.5	27.3	29,350	129.6	23.5
1942[b]	43,000	126.8	1.7	625,000	187.4	31.6	45,000	198.7	53.3
Total[c]	....	....	26.8	....	....	87.4	....	....	98.7

[a] Percentage increase over the preceding year. [b] Estimated. [c] Percentage increase, 1942 over 1939.

The progressive effect of the increasing traffic load is shown by the percentage data in Cols. 3, 6, and 9, Table 10. Comparing 1942 with 1939, the cumulative freight traffic increase is shown as 26.8% in carloadings and 87.4% in ton-miles. The greater increase in ton-miles is due to the increased load per car and an increase in the length of haul. The average load per car in 1942 will be more than 4 tons greater than the average load in 1939, whereas the average haul per ton of freight in 1939 was 351 miles, and today is about 420 miles. In the

[24] "Transportation on the Home Front," by Ralph Budd, *Civil Engineering*, September, 1942, p. 503.

passenger service, including troop movements, the number of revenue passengers carried one mile nearly doubled in 1942 as compared with 1939.

Table 11 shows the progress made by the railroads since September 1, 1939, in freight-car and locomotive ownership. It shows the net results the railroads

TABLE 11.—RAILROAD OWNERSHIP OF FREIGHT CARS AND LOCOMOTIVES

Date; September 1:[a]	Freight Cars				Locomotives			
	Total owned	Unserviceable		Serviceable	Total owned	In Bad Order		Serviceable
		No.	%			No.	%	
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1939	1,650,423	224,603	13.6	1,425,820	42,848	8,381	19.6	34,467
1940	1,640,924	137,869	8.4	1,503,055	41,608	6,268	15.1	35,340
1941	1,648,502	78,044	4.7	1,570,458	41,384	4,247	10.3	37,137
1942[a]	1,736,909	53,811	3.1	1,683,098	41,566	2,735	6.6	38,831

[a] Except 1942, which was August 15 for freight cars and August 1 for locomotives.

have obtained from installation of new equipment and the repair of existing equipment. The main achievement has been in the reduction in number and proportion of unserviceable cars and locomotives.

The number of freight cars in bad order on September 1, 1939, was 224,603, or 13.6% of total ownership. This was reduced to 53,811 freight cars on August 15, 1942, or 3.1% of total ownership. Thus, although the total ownership of freight cars increased by only 86,486 cars, the number of serviceable units increased by 257,278 cars.

In the case of locomotives, the ownership on August 1, 1942, was less than on September 1, 1939, by 1,282 locomotives. On the other hand, the number of serviceable engines increased by 4,364 units, those in bad order having been decreased from 8,381 on September 1, 1939, to 2,735 on August 1, 1942.

TABLE 12.—IMPROVEMENT IN THE UTILIZATION OF RAILWAY EQUIPMENT

Year	Freight Cars		Freight Locomotives		Passenger Locomotives	
	Ton-miles[a]	Index No.	Ton-miles[b]	Index No.	Car-miles[c]	Index No.
1939	692	100.0	167,870	100.0	1,408	100.0
1940	721	104.2	178,344	106.2	1,462	103.8
1941	833	120.4	198,997	118.5	1,532	108.8
1942[d]	945	136.6	214,544	127.8	1,615	114.7

[a] Freight-car utilization, in net ton-miles per active freight-car day. [b] Freight-locomotive utilization, in gross ton-miles per active freight-locomotive day. [c] Passenger-locomotive utilization, in passenger-train car-miles per active locomotive day. [d] First six months of 1942.

It is clear, however, that the railroads could not have met the heavy traffic demands of today without improved performance. Table 12 shows the steady improvement in utilization of equipment that has occurred since 1939. With respect to freight-car utilization, the average car in 1942 has been equal to

more than one and one-third cars in 1939. Freight-locomotive utilization has increased 27.8%, and passenger-locomotive utilization has increased 14.7%. The railways have been doing a good job.

Table 8 shows that the relative volume of traffic for the year 1939 aggregated 543.4 billion revenue freight ton-miles and 40 billion revenue passenger-miles. It is estimated from data in Table 10 (which were furnished by the Association of American Railroads) that railway traffic will be increased 291.6 billion revenue ton-miles, or 87.4%, and revenue passenger-miles will be increased 22.3 billion miles, or 98.7%, in 1942 compared with 1939. Reliable information is not available (as of December, 1942) as to just what the traffic will be for other carriers; however, information at hand indicates that the traffic of these agencies also will be materially increased over that of 1939. It is obvious that changed conditions due to the war effort have greatly increased the demand for transportation by the highway, waterway, pipe line, and air carriers, as well as by the railroads. All of these agencies are being handicapped by the lack of equipment, and it now seems that the gasoline shortage in the East and the rubber-tire situation may have a retarding effect on the capability of the highway carriers to continue to handle the volume of traffic now prevailing.

As previously stated, the inland waterways have been improved and modernized during the past quarter of a century. The improvements of some of them were projected in the feverish rush of World War I. World War II threatens to start a similar expansion of waterway improvements. As illustrations, construction of the Florida Ship Canal, the Lake Erie-Ohio River Canal, and the St. Lawrence Deep Seaway—each possessing strong political and commercial supporters—is being advocated earnestly by their backers, either as absolutely necessary to the war effort or as desirable "make work projects" as soon as the war is over. Therefore, it seems to the writer that an examination of previous waterway developments is appropriate at this time, and in particular the Ohio River waterway system, which is probably the most complete of any improved waterway system in the United States.

This was a subject given consideration by Committee XVI—Economics of Railway Location and Operation—of the American Railway Engineering Association (A.R.E.A.) in 1940; the following conclusions[25] were formulated at that time:

> "1—The Ohio River navigation system consists of 3,612 route miles (981 on the Ohio river between Pittsburgh and Cairo, and 2,621 on tributary streams), and represents an investment on the part of the taxpayers of the United States of some $234,000,000, and a total expenditure (as of January 30, 1938) including maintenance and a relatively small amount of unexpended appropriations, of $362,000,000. Based on the operation during 1937, a total of about 4,205,000,000 ton-miles of freight was transported on these rivers, a large percentage of which would have moved on the railroads had government improved waterways not been available.
>
> "The annual cost of transportation on these rivers to the nation is estimated at approximately $34,000,000, of which about one-half is paid or absorbed by the taxpayers and the other half by the users of the waterways.

[25] *Proceedings*, A.R.E.A., Vol. 41, 1940, p. 149.

"There is no evidence of record that the users of the waterways pass on to the public in general the savings that accrue to them as compared with the rail freight rate. Waterway transportation may be cheap to the favored shipper but is costly to the taxpayers, who foot the bill for improving, operating and maintaining the locks, dams and channels."

A more recent study of this subject has been reported by C. Emery Troxel,[26] who develops some very interesting cost figures (excluding the cost paid by the shipper) representing the cost per ton-mile of all traffic on this waterway system which is borne by the government or taxpayers.

The facts presented raise questions as to the effectiveness of waterways (compared to railways) as a means of maximizing wartime transportation, and of the economic justification of most, if not all, of such enterprises at any time. Concerning these questions Professor Troxel states:

"The presence of the Ohio Waterways thereby may relieve the pressure on railroads and improve the war effort of the country. Yet this gain would be a sort of windfall for the nation, for these waterways were developed more for peacetime than for wartime use. Such additional transport capacity, granted that it may prove convenient, might have been provided more economically by other means. * * * It is customary for waterway protagonists to acclaim the railroad-rate reductions which follow the opening of a waterway. Though they correctly note the transportation expense reduction for some commodities and companies, they fail to recognize the possibility of contrary consequences elsewhere. Perhaps the rate reductions to meet water competition have the effect of forcing rate increases for commodities or areas where there is less competition. Or, in the event of a continuance of deficit operations, railroads may be forced to abandon parts—the weaker branches—of their systems. Communities near the waterway or elsewhere—for toll-free water transportation may provoke abandonments at inland points—may be compelled to make more expensive (for some goods) use of trucks. These effects may appear slowly, and yet they may be inevitable."

Under date of September 18, 1942, the Board of Investigation and Research transmitted its annual report to the President and the Congress of the United States, in accordance with Section 305 of Part 1, Title III, Transportation Act of 1940. In this first report the Board outlines the scope of its investigations, which include the flow of traffic, the cost and efficiency of transportation services, amount of public aids received and taxes paid by each type of carrier, labor relationships, shipper-carrier relationships, regulatory policy and practices, and interterritorial freight rates. The following statement[27] is of particular significance:

"The coming of the war has imposed extraordinary burdens upon the national transportation system. Within a very brief period carriers which for years had suffered from insufficient traffic were obliged to expand their services to the limits of their capacity, and danger signs warning of critical shortage of transportation began to appear. This sudden shift has not, however, altered the fundamental problems of transportation, nor has it changed the essential characteristics of the various types of carriers; rather, it has focused attention upon certain aspects of these

[26] "How 'Cheap' Is River Transport?" by C. Emery Troxel, *Railway Age*, September 12, 1942, p. 408.
[27] Annual Report of Board of Investigation and Research, September 22, 1942, p. 3.

problems, demonstrating the need for adequate capacity and flexibility in transportation and making more certain than ever the prospect of far-reaching readjustments in our transportation facilities.

"These emergency conditions have not diminished the significance of the problems which the Board is directed to investigate. This was recognized by the President when, by proclamation of June 26, 1942, he extended the term of the Board to its statutory limit, September 18, 1944. The proclamation declares that—an efficient transportation system is essential to the nation in peace and war, and the national interest requires the development of informed policies by which such a system may be promoted, strengthened and maintained."

The Board of Investigation and Research reports progress in its studies of relative economy and fitness, public aids, taxation, and interterritorial freight rates. Under the caption "Further Reports" appears the following:

"The Board is proceeding as rapidly as possible with the research contemplated by the act. The entry of this country into the war has complicated the problems which the Board is directed to study, but has not changed their essential character. The scarcity of trained personnel and the burdens imposed upon the carriers by wartime demands have in some ways increased the difficulty of the Board's investigations. Yet the war has emphasized the importance of the work and has facilitated it in some important respects. It has made available recent data concerning the capacity of our transportation facilities. It has already brought forth new operating methods and has tested expedients which otherwise could be dealt with only in theory. These experiences are of great value in considering methods of developing a national transportation system adequate for the present and future needs of commerce and national defense.

"The Board's studies of the interterritorial freight rate structure will be filed in January 1943; its report on public aids to carriers will be filed early in 1943; its report on carrier taxation will be completed by the end of the current fiscal year. Its investigations of relative economy and fitness of carriers are of such magnitude and importance as to require a longer period for completion. These further reports will contain the full findings and recommendations of the Board."

The finding and recommendations of the Board will be of national interest and may have an important bearing on the future policy affecting the national transportation economy.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

HYDRODYNAMICS OF MODEL STORM SEWER INLETS APPLIED TO DESIGN

Discussion

BY G. S. TAPLEY, ASSOC. M. AM. SOC. C. E.

G. S. TAPLEY,[15] Assoc. M. Am. Soc. C. E.[15a]—The problem attacked in the paper is of a type upon which little has yet appeared in literature; features believed to be novel were included. It is regretted, therefore, that criticism, either for or against, was not more widespread.

Professor Powell's comments are very much to the point in several particulars. His statement that the method of distributional analysis seems worthy of study is believed to be particularly pertinent when, as in the case of storm-sewer inlets, the lateral distribution is so characteristically a feature of the phenomenon.

The study of distribution itself is also an active field at present. Attention is invited to the observed, practically perfect independence of the values of κ from the total flow Q_t in the model channel. Table 9 shows, for comparison, values of κ representing $S_f = 0.02$, or $Q_t = 0.0615$, and corresponding values of κ representing the average for $S_f = 0.01$, 0.02, 0.04, and 0.06, or $Q_t = 0.0434$, 0.0615, 0.0870, and 0.1066. All data represent $a_0 = 0.0460$, an average obtained from a large number of observations.[16]

TABLE 9.—COMPARISON OF VALUES OF κ

Description	Values of z_i							
	0.168	0.339	0.508	0.674	0.841	1.003	1.257	1.700
κ (S_f=0.02)	0.278	0.517	0.690	0.805	0.882	0.928	0.969	1.001
κ (average)	0.280	0.518	0.693	0.810	0.886	0.932	0.970	1.000

NOTE.—This paper by G. S. Tapley, Assoc. M. Am. Soc. C. E., was published in March, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings* as follows: September, 1942, by Ralph W Powell, M. Am. Soc. C. E.

[15] Junior Civ. Engr., Bureau of Eng., City of Los Angeles, Los Angeles, Calif.

[15a] Received by the Secretary October 28, 1942.

[16] Values in Table 9 are from "Depth and Distribution Data for Flow in Model Street," by G. S. Tapley, Table 19. The original manuscript is on file at Engineering Societies Library, 29 West 39th Street, New York, N. Y.

Inasmuch as Professor Powell calls attention to the writer's use of the momentum average, the following remarks upon average velocities may be in order: The so-called average velocity, "discharge divided by area," has been aptly called the "apparent average" velocity. It should be kept in mind that this velocity is an average one only in the uninteresting case where velocity does not vary throughout the cross section. The following mathematical expression shows this:

$$v' = \frac{\int_0^A v\,dA}{\int_0^A dA} \qquad (55)$$

in which A represents area. Moreover, the physical conception is not clear; a particle of fluid cannot be described satisfactorily by its area of cross section, dA.

On the other hand, the expression for the momentum-average velocity, $\bar{v}$, is one for which the physical conception is grasped readily:

$$\bar{v} = \frac{\int v\,dM}{\int dM} = \frac{\int v\,\rho\,dQ}{M} = \frac{\rho \int_0^A v^2\,dA}{M} \qquad (56)$$

in which M represents mass per second, and Q represents volume per second. Each elementary mass of fluid is weighted by its particular velocity. The average, $\bar{v}$, is the velocity of all the particles moving as one mass when the corresponding momentum equals the aggregate of the momentums of all the individual masses moving at velocity v.

The expression for the energy-average velocity, $\bar{\bar{v}}$, may be obtained from

$$\bar{\bar{v}}^2 = \frac{\frac{1}{2}\int v^2\,dM}{\frac{1}{2}\int dM} = \frac{\int v^2\,\rho\,dQ}{\int \rho\,dQ} = \frac{\rho \int_0^A v^3\,dA}{\int_0^Q \rho\,dQ} \qquad (57)$$

Hence,

$$\bar{\bar{v}} = \frac{\left(\rho \int_0^A v^3\,dA\right)^{0.5}}{\left(\int_0^Q \rho\,dQ\right)^{0.5}} \qquad (58)$$

In Eq. 58 the physical conception is again not so clear as in the case of $\bar{v}$, because the denominator represents the square root of a mass per second. It is required, then, to state: $\bar{\bar{v}}$ is the velocity at which the square root of the mass per second moves when the energy of the total mass equals the sum of the energies of the individual particles.

Assuming that both energy-average and momentum-average velocities exist, the writer believes that the criterion for choice between them should be: Use the energy-average velocity when the phenomenon is properly and conveniently expressible in terms of work and energy; and use the momentum-average velocity when momentum and force are the proper mediums for expression.

Reference should be made to a bulletin of the National Bureau of Standards, dated July, 1942, regarding energy and momentum methods.[17] The bulletin calls attention to the long-standing nature of the controversy over the methods, and states that Boussinesq advocated the momentum method, whereas Coriolis advocated the energy method, Coriolis' view generally prevailing. The bulletin further gives the view of G. H. Keulegan that either method is correct, provided the proper interpretation is placed upon the operation, but that, practically, it is preferable to use the momentum method.

It should be emphasized that the ratio $\frac{\bar{v}}{v'}$, mentioned by Professor Powell, represents the ratio of the momentum-average velocity of the inlet flow to the discharge-over-area average velocity of the total flow. These quantities, therefore, are not comparable in the manner Professor Powell evidently believes they are, since they do not represent the same cross sections of flow.

In an investigation of backwater effect,[18] the writer found that the relationships in Table 10 hold for the total width of cross section, the area of cross section being 0.0298 sq ft. The ratio is thus constant, but less in magnitude than computed by Professor Powell. The writer did not feel warranted in ignoring a constant discrepancy even as small as 6%, because the quantity under investigation (Reynolds' number) was expected to make itself evident by only a feeble effect within the model range.

TABLE 10.—RELATIONSHIP BETWEEN $\bar{v}$ AND v'

Description	Values of S_f			
	0.01	0.02	0.04	0.06
$\bar{v}$	1.55	2.18	3.07	3.77
v'	1.46	2.06	2.90	3.57
$\bar{v}/v'$	1.06	1.06	1.06	1.05

Professor Powell inquires whether Eq. 54 could not have been used as well as Eq. 6*b*. The following arguments seem to prove that it could not, although some of the facts involved could not be anticipated without regard to the experimental data. All quantities of physics are products,[19] and any set of observations can be represented by a potential series.[20] Moreover, a product of a series is only another series. Eq. 54 evidently would apply to a series in which there were no terms such as **FR**; such terms are required in the experimental case because **F** and **R** vary simultaneously. In a similar case, Morrough P. O'Brien and George H. Hickox,[21] Members, Am. Soc. C. E.,

[17] *Technical News Bulletin*, National Bureau of Standards, July, 1942, p. 50.

[18] "Inlet Flow with Backwater Effect," by G. S. Tapley, Table 9. The manuscript is filed with Engineering Societies Library, 29 West 39th Street, New York, N. Y.

[19] "Mathematics of Modern Engineering," by Robert E. Doherty and Ernest G. Keller, John Wiley & Sons, Inc., New York, N. Y., Vol. 1, 1936, pp. 140 and 145.

[20] "Engineering Mathematics," by Charles P. Steinmetz, McGraw-Hill Book Co., Inc., New York, N. Y., 1917, p. 212.

[21] "Applied Fluid Mechanics," by Morrough P. O'Brien and George H. Hickox, McGraw-Hill Book Company, Inc., New York, N. Y., 1937, p. 99.

state that the unknown function should be considered as the product of two infinite series in the dimensionless groups.

Professor Powell is correct in stating that the meaning of h in connection with its use in $\mathbf{F}$ and $\mathbf{R}$ was not sufficiently emphasized. The dimension, of course, is the one selected as being descriptive of the characteristic phenomenon, gravitation, and it represents the fall of the water during its transit over the effective part of its course. Had viscosity been the principal consideration, another dimension would have been selected. When both gravity and viscosity are considered important, a single dimension is used in both the Froude and Reynolds numbers, but the dimension best representing the most important force is the one selected.

The writer agrees that it might have been preferable to use a larger model, but for the reason that difficulty was experienced in measuring depths accurately at the approach section, and not because of fear regarding the effect of surface tension. Considerable attention was given to the possible effect of surface tension because of the small size of the model, but it was concluded that the effect was inappreciable except in a few minor cases, such as when the overflow was approaching zero magnitude. However, in the case of submerged inlets (a subject not included in the paper), where the inlet operates as an orifice, surface tension was deemed to be an important consideration, and in an unpublished report entitled "Submerged Curb-Opening Storm Drain Inlets," the writer included it in the analysis by means of which the orifice coefficient was derived. The principal argument against a larger model, of course, is that time and expense items increase rapidly with increase in size. With a larger model it is possible, moreover, that it would have been difficult to have determined the form of the function of $\mathbf{R}$, because the effect of viscosity becomes feebler with increase in size.

Backwater Effect.—Subsequent to completion of the analysis of free flow, an investigation of backwater effect[18] was undertaken. The following results of the latter investigation are submitted here as being substantiative, to some extent, of the correctness of the former analysis, and also for the value they may have in extending the range of conditions covered.

Following Eq. 36a, let

$$J = \frac{K\,\mathbf{F}}{0.230 - \dfrac{958}{\mathbf{R} + 4{,}510}} = \phi(\bar{v}, b, g, h) \qquad (59)$$

In Eq. 59, b is the characteristic linear dimension representing backwater effect, and equals the difference in elevation between the surface of the backwater body (assumed level) and the lip of the inlet at its center. The effect of viscosity has been assumed to be negligible in the backwater phenomenon, because that phenomenon is characteristically one of shock, rapid change of elevation of the surface, or wave motion.

Figs. 18(a) and 18(b), respectively, show experimental values of J as functions of b and $\bar{v}$. Fig. 19 shows values of $\bar{v}$ versus b for constant values of J and for $n = 0$—that is, no overflow. The locus for $J = 1$ represents the limit-

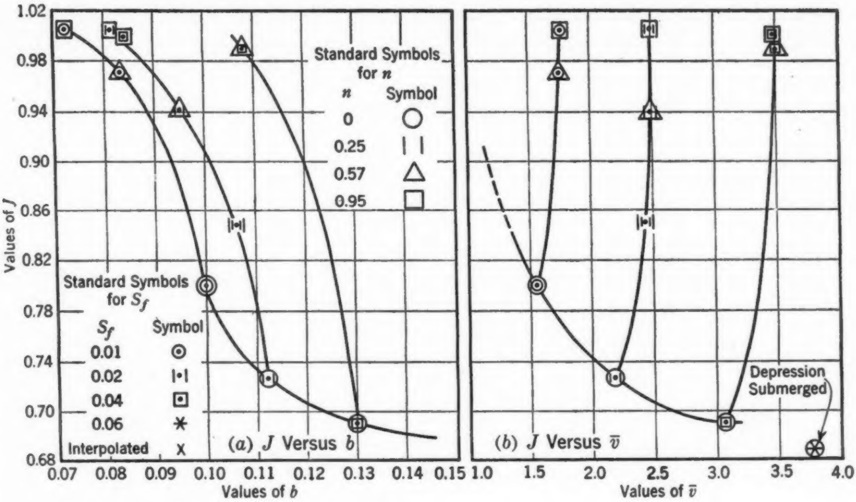


FIG. 18.—VALUES OF J AS FUNCTIONS OF b AND $\bar{v}$ ($a_0 = 0.0460$)

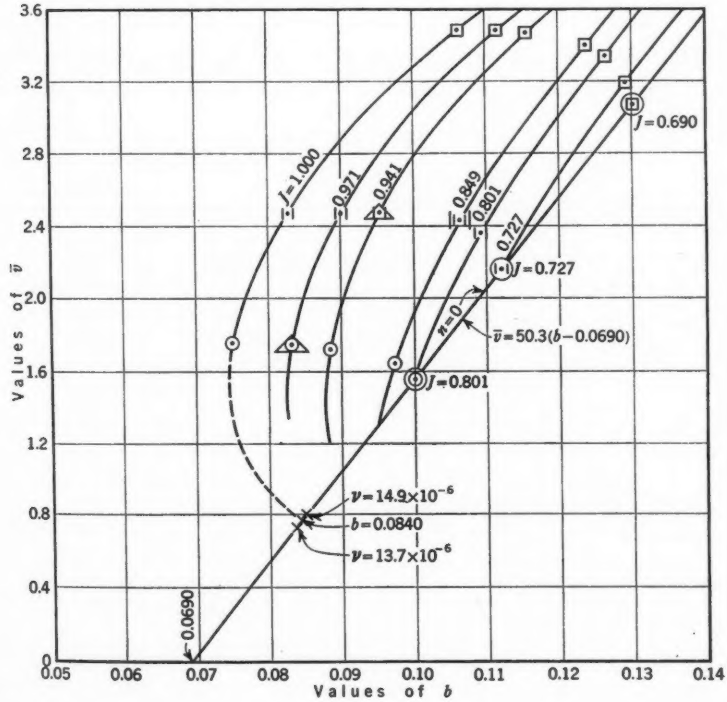


FIG. 19.—EXPERIMENTAL VALUES OF $\bar{v}$ AS A FUNCTION OF b, FOR CONSTANT VALUES OF J AND FOR $n = 0$

ing condition between free flow and backwater effect. The locus for $n = 0$ is a straight line whose equation is

$$\bar{v} = 50.3\,(b - 0.0690) \qquad (60)$$

Values of the coordinates of the point of intersection of the loci for $n = 0$ and $J = 1$ were computed from Eq. 59, using the constant value

$$K\,\mathbf{F} = K_t\,\mathbf{F}_t = 0.133 \qquad (61)$$

and an average experimental value of ν.

It can be reasonably anticipated that the arguments in the function ϕ will combine in such a manner that J is expressed as a function of a Froude number and of a number expressing a geometrical dimension of the backwater body. It was found, however, that the characteristic linear dimension that enters these numbers is a difference of depths rather than a depth such as b.

The simplest case of flow is that for which $n = 0$, because in that case no overflow exists, and the width at the approach section of the filament of flow entering the inlet is the total width of the channel—z_i has the constant value z_t. Fig. 20 shows values of J versus various powers of $\frac{1}{B_\alpha}$, in which

$$B_\alpha = \frac{b - 0.0690}{h} \qquad (62)$$

In Eq. 62, 0.0690 is the value of b when $\bar{v} = 0$ and $n = 0$, as shown in Fig. 19. The equation for J in terms of B_α has the simple form

$$J = 0.653 + 0.0139\left(\frac{1}{B^2_\alpha}\right) \qquad (63)$$

Note that in Fig. 20 the data representing $S_f = 0.06$ correspond to the condi-

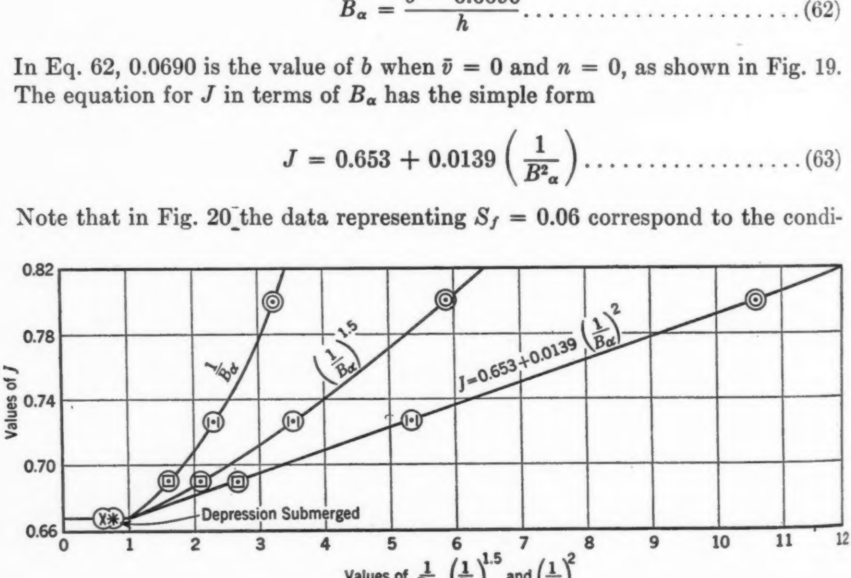


FIG. 20.—EXPERIMENTAL VALUES OF J AS A FUNCTION OF VARIOUS POWERS OF $\frac{1}{B_\alpha}$

tion wherein the backwater wave has reached a point upstream far enough to submerge the depression completely, so that the velocity distribution at the approach section is no longer normal for the value of S_f.

It was found that, for conditions of flow other than those representing $n = 0$, other terms than B_α are necessary. The results are shown in Fig. 21 and are expressible by means of the linear relationships:

$$J - \mathbf{F}^{0.5}{}_\Delta B^{1.5}{}_\Delta = 0.867 - 2.963 (B^{1.5}{}_\Delta - 0.0170) \qquad (64a)$$

for values of b greater than 0.0840, and

$$J - \mathbf{F}^{0.5}{}_\Delta B^{1.5}{}_\Delta = 0.867 + 0.544 (- B_\epsilon)^{0.5} \qquad (64b)$$

for values of b less than 0.0840. In Eqs. 64: $\mathbf{F}_\Delta = \frac{\bar{v}^2}{g \Delta b}$; $\Delta b = b - 0.0774$; $B_\Delta = \frac{\Delta b}{h}$; $B_\epsilon = \frac{\epsilon b}{h}$; and $\epsilon b = b - 0.0840$. (Note that, by Eq. 61, the quantity $0.867 = 1 - K_t \mathbf{F}_t$.)

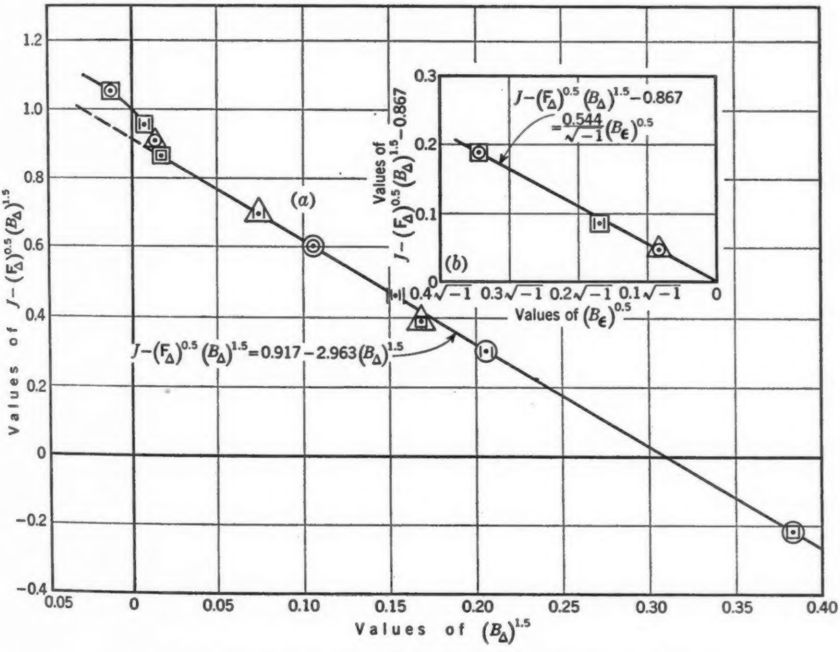


FIG. 21.—FLOW FACTORS FOR $b > 0.0840$ AND $b < 0.0840$ ($a_0 = 0.0460$)

The difference in the parameters 0.0774 and 0.0840 is 0.0066, which is the fall in half the length of the inlet due to the normal slope of the model. This fact suggests the manner in which the length of inlet and slope of street enter the backwater function and should be useful when, later, these quantities are investigated as variables. It can be shown readily that

$$B_\Delta = B_\epsilon + B_{SX} = B_\epsilon + \frac{0.0066}{h} \qquad (65a)$$

and

$$\mathbf{F}^{0.5}{}_{\Delta}\, B^{1.5}{}_{\Delta} = \pm\, \mathbf{F}^{0.5}{}_{\epsilon}\, B^{1.5}{}_{\epsilon} + \mathbf{F}^{0.5}{}_{SX}\, B^{1.5}{}_{SX} \quad (65b)$$

Eqs. 65 inserted in Eqs. 64 will yield more general expressions. Note that $\mathbf{F}^{0.5}{}_{SX}\, B^{1.5}{}_{SX}$ varies only with $\bar{v}$, and $\mathbf{F}^{0.5}{}_{\epsilon}\, B^{1.5}{}_{\epsilon}$ varies with both $\bar{v}$ and b. The latter term ($\mathbf{F}^{0.5}{}_{\epsilon}\, B^{1.5}{}_{\epsilon}$) thus corresponds to the term $B^2{}_{\alpha}$ of Eq. 63. The terms of Eq. 65*b* are converted easily into expressions for moment of velocity, such as

$$\mathbf{F}^{0.5}{}_{\Delta}\, B^{1.5}{}_{\Delta} = \frac{\bar{v}\, \Delta b}{g^{0.5}\, h^{1.5}} \quad (66)$$

if g and h are assumed not to be dimensional constants.

In conclusion, the writer submits the good correlation, the rationality, and the simplicity in the foregoing paragraphs as being good evidence that the expressions for flow derived in the main text of the paper properly account for all factors which influence the flow, including those expected to be the most troublesome—distribution and viscosity.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FUNDAMENTAL ASPECTS OF THE DEPRECIATION PROBLEM

A SYMPOSIUM

Discussion

BY KERNAN ROBSON, ESQ.

KERNAN ROBSON,[54] ESQ.[54a]—In the Symposium, the treatment of the depreciation problem seems concerned mainly with plant and equipment. From the point of view of the average realtor who makes depreciation deductions from tens of billions of dollars of depreciable improvements on real estate, there are many other factors to be weighed in addition to those proposed by the authors.

Although the writer will not discuss the mathematics of these factors, he wishes to emphasize the immensely important tax problems involved. These problems will be greatly accentuated by the 5-yr depreciation program applied to billions of dollars worth of war structures.

One of these problems may be called the "Problem of Annual Low Brackets Against Total High Brackets When Profit or Loss is Ascertained."

An important inequity resulting from the present treatment of depreciation and depletion reserves in adjusting, annually, the cost or other base of property is the following:

The deduction from net income for such reserves as the taxpayer is allowed from year to year is in the lower brackets; but upon sale or other disposition of these assets, resulting in gain or loss, the total of depreciation or depletion reserves theretofore taken is segregated into one sum which brings the taxpayer into a higher bracket than those in which his income put him in the years these

NOTE.—This Symposium was published in November, 1941, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: November, 1941, by Messrs. Edwin F. Wendt and L. T. Fleming, and Anson Marston; December, 1941, by Messrs. William G. Atwood, George E. Goldthwaite, Nathan B. Jacobs, H. J. Flagg, Nelson Lee Smith, and C. Beverley Benson; February, 1942, by Messrs. Paul T. Norton, Jr., H. L. Ripley, and Carroll A. Farwell; March, 1942, by Messrs. Thomas R. Agg, Conde B. McCullough, W. V. Burnell, Roger L. Morrison, Wallace B. Carr, W. L. Waters, A. G. Mott, David A. Kosh, E. G. Walker, and K. Lee Hyder; April, 1942, by Messrs. James T. Ryan, John C. Page, and John S. Worley; June, 1942, by Messrs. John F. Miller, and Fred Asa Barnes; and September, 1942, by Robert F. Legget, Assoc. M. Am. Soc. C. E., and Erwin E. Hart, Esq.

[54] Real Estate and Investments, San Francisco, Calif.

[54a] Received by the Secretary November 2 ,1942.

reserves were taken. An example used as far back as 1938 began with the assumptions that:

(*a*) A taxpayer has a plant which he bought for $100,000;
(*b*) The allowed depreciation was 10% per year for ten years;
(*c*) The taxpayer had no other net taxable income at the end of each of the ten years;
(*d*) The tax rate remained the same throughout the 10-yr period; and
(*e*) At the end of the ten years, the taxpayer sold the property for the original cost price ($100,000).

He would then have an accumulated depreciation reserve of $100,000 on which the total taxes—federal and State of California (1938)—would be $41,950. However, the total tax savings (and thus the total sum withheld and due the government by reason thereof) would have been $8,500. That is all he really owes the government by reason of his withdrawal of the allowed or allowable annual depreciation from annual income. The government would thus make a profit of $33,450.

Again, if the taxpayer were to pay someone, at the end of the 10-yr period, $100,000 to take the property off his hands, the $100,000 would not be a loss but would actually be matched by the accumulated depreciation.

The case of Kennedy Laundry Company vs. the City of San Francisco (Case 46 BTA, No. 9, January 14, 1942) is one of many that set forth amongst other matters the principle as to the adjustment of the cost basis by depreciation or depletion allowed or allowable. In substance this principle may be stated as follows (46 BTA No. 9, Section 19.113 (b) (1) 1):

> The cost or other basis of property value to which depreciation or depletion is applicable shall be decreased annually, in addition to all the other deductions, by an amount allowed or allowable due to the exhaustion, wear and tear, obsoleteness or depletion sustained by said property.

It is the condition defined in the general rule that brings about the inequity to which the writer refers herein.

Under that rule such deductions result in the taxpayer's being in the lower brackets when those deductions are made, but when property is disposed of and gain or loss is ascertained, the taxpayer is brought into the upper brackets. Thus, whether the taxpayer makes a gain or a loss, the government gains more (in many instances excessively more) than it is entitled to gain.

This entire doctrine that cost or the adjusted basis shall be reduced annually by the allowable or allowed withdrawals from net income of depreciation or depletion reserves is based on both a mistaken business theory and a mistaken accounting theory.

From a business standpoint, the cost price or sales price of property is entirely different from the reserves that may be set up by reason of depreciation or depletion. Correct accounting which seeks to reflect the true business process could scarcely support a practice in which a debtor, at the end of a transaction, receives a larger sum than has been withheld from him or than he could show, to that time, as belonging to him.

The writer therefore suggests, as a means of correcting the inequity herein set forth, that the taxpayer's books should be set up as follows:

Every year in which depreciation or depletion is allowed or allowable, there should be set up what might be denominated a "suspended reserves account."

Into this account would be entered the difference between the amount the taxpayer paid in income taxes and the amount he should have paid, had no deductions from net income for these reserves been made. When the property is finally sold, or otherwise disposed of, so that gain or loss is ascertained, these annual entries adjusted by charges against them for repairs and replacements (not maintenance or upkeep, which goes to operating costs, and additions or betterments which go to capital costs), added together would constitute the sum of money due the government. The difference between the base of the cost of the property plus or less other proper adjustments (not depreciation or depletion deductions) and the selling price is a profit or a loss in that transaction.

This is an entirely different transaction from the setting up of annual reserves and the fixing of the contingent or final interest of the government therein.

The profit or loss to the taxpayer in the first instance should not be confused or mixed up in any way with the gains to the government which would be set forth in this "suspended reserves account."

The writer considers that "obsolescence" is probably the proper term for the continuous process, and "obsoleteness" for its completion. The Bureau of Internal Revenue, U. S. Treasury Department, defines obsolescence as:

That impairment of depreciable property brought about

(1) By progress of the arts;
(2) Effluxion of time; and
(3) Change of use or useful life.

Of these factors, Item (3) is vastly the most important. The writer knows of no all-embracing rule that can be formulated to treat it, either for equipment, plant, or structure.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

MOMENT BALANCE: A SELF-CHECKING ANALYSIS OF RIGIDLY JOINTED FRAMES

Discussion

BY JOHN MASON, ESQ.

JOHN MASON,[21] ESQ.[21a]—There are two points suggested by this most interesting paper, upon which the writer would like to comment:

(1) Why introduce a new concept "X" in addition to the fixed-end moment M_F? The fixed-end moment must be found as a guide to the estimate of the end moment, and it will be shown that it can be re-used just as easily in the moment balance as the new expression.

(2) Any saving in labor by this method depends on a reasonable initial estimate of the end moments which, it is submitted, is more than offset by the certainty of working methodically to the result by methods that do not depend on any such estimate.

Using the method of "allocated moments" presented by the writer[22] in 1940, an alternate solution of the example given in Table 2 (using the same sign convention and notation) is given in Table 7.

The essential feature of the method of allocated moments consists in finding the angular rotation of the joint or, for convenience, a value called the "joint moment," which is a measure of the angular rotation of the joints, being related to it by the equation:

$$\text{Joint rotation} = \frac{\text{Joint moment}}{4 \times \text{joint stiffness} \times \text{E}} = \frac{\text{Joint moment}}{4\,E\,\Sigma K} \quad \ldots\ldots (22)$$

in which the "stiffness of the joint" equals the sum of the stiffness of all members radiating from that joint.

NOTE.—This paper by R. J. Cornish, Esq., was published in May, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: May, 1942, by Messrs. D. D. Matthews, and William A. Larsen; June, 1942, by Messrs. A. A. Eremin, L. E. Grinter, and B. J. Aleck; September, 1942, by Messrs. Frederick S. Merritt, and Ralph W. Stewart; October, 1942, by G. W. Stokes, Esq.; and November, 1942, by Messrs. Bruce Jameyson, R. E. Bowles, and William Morris.

[21] Cons. Engr., London, England.

[21a] Received by the Secretary October 5, 1942.

[22] *Journal*, The Inst. of Structural Engrs., April, 1940, p. 577; for discussion, *ibid.*, March, 1941, p. 43.

The joint moment is found by taking the sum of the fixing moments and adding algebraically to this value a series of allocated moments, each allocated moment being equal to $-\frac{1}{2}$ the adjacent joint moment distributed in proportion to the stiffness of the members at that joint. The allocated moments are the

TABLE 7.—COMPUTATIONS FOR METHOD OF ALLOCATED MOMENTS

Allocation	Joint A			Joint B				Joint C		
	Member AD	Member AB	Joint moment	Member BA	Member BE	Member BC	Joint moment	Member CB	Member CF	Joint moment
K	3	5	16	5	4	5	28	5	3	16
M_F	....	−8.4	−8.4	+2.8	....	....	+2.8	....	....	0
First	....	....	....	+2.6	+1.1	0	+3.7	....	....	....
Second	−0.4	−1.2	−1.6	....	....	....	....	−1.2	−0.8	−2.0
Third	....	....	....	+0.5	+0.3	+0.6	+1.4	....	....	....
Fourth	−0.1	−0.2	−0.3	....	....	....	....	−0.2	−0.1	−0.3
Fifth	....	....	....	+0.1	+0.1	+0.1	+0.3	....	....	....
Sixth	−0.0	−0.1	−0.1	....	....	....	....	−0.1	−0.0	−0.1
Sum	−0.5	−9.9	−10.4	+6.0	+1.5	+0.7	+8.2	−1.5	−0.9	−2.4
Balance	+3.9	+6.5	....	−2.9	−2.4	−2.9	....	+1.5	+0.9	....
M	+3.4	−3.4	....	+3.1	−0.9	−2.2	....	0.0	0.0	....

TABLE 7.—(*Continued*)

Allocation	Joint D				Joint E					Joint F			
	Member DA	Member DG	Member DE	Joint moment	Member ED	Member EB	Member EH	Member EF	Joint moment	Member FE	Member FC	Member FK	Joint moment
K	3	4	6	26	6	4	5	6	42	6	3	4	26
M_F	...	...	...	...	...	...	...	−11.5	−11.5	+5.2	...	...	+5.2
First	+1.6	0.0	+1.6	+3.2	...	...	...	....	....	+1.6	0.0	0.0	+1.6
Second	...	...	...	...	−0.7	−0.9	0.0	−1.6	−3.2	...	...	...	...
Third	+0.3	0.0	+0.5	+0.8	...	...	...	....	....	+0.5	+0.4	...	+0.9
Fourth	...	...	...	...	−0.2	−0.2	...	−0.2	−0.6	...	...	...	...
Fifth	0.0	...	+0.1	+0.1	...	...	...	....	....	+0.1	0.0	...	+0.1
Sixth	...	...	...	...	0.0	0.0	...	0.0	....	...	...	...	...
Sum	1.9	0.0	+2.2	+4.1	−0.9	−1.1	0.0	−13.3	−15.3	+7.4	+0.4	...	+7.8
Balance	−0.9	−1.3	−1.9	...	+4.4	+2.9	+3.6	+4.4	....	−3.6	−1.8	−2.4	...
M	+1.0	−1.3	+0.3	...	+3.5	+1.8	+3.6	−8.9	....	+3.8	−1.4	−2.4	...

same as the "carry-over" moments in "moment-distribution" methods. In the concise method on which Table 7 is built, however, the moments totaling +3.7 (allocated to joint B in the first allocation) are first added to the fixed-end moment, +2.8, and the sum, +6.5, is allocated in bulk to the adjacent joints.

Once the rotation of the joint has been established, the end moments follow from the slope-deflection relationships:

$$M_{ON} = M_{F(ON)} + 4\,E\,K_{ON}\,\phi_O + 2\,E\,K_{NO}\,\phi_N$$

$$= M_{F(ON)} + K_{ON} \times \frac{\text{Joint moment}_O}{\Sigma K_{ON}} + \frac{K_{NO}}{2} \times \frac{\text{Joint moment}_N}{\Sigma K_{NO}} \quad \text{(23a)}$$

$$M_{NO} = M_{F(NO)} + 4\,E\,K_{NO}\,\phi_N + 2\,E\,K_{ON}\,\phi_O$$

$$= M_{F(NO)} + K_{NO} \times \frac{\text{Joint moment}_N}{\Sigma K_{NO}} + \frac{K_{ON}}{2} \times \frac{\text{Joint moment}_O}{\Sigma K_{ON}} \quad \text{(23b)}$$

or, in general,

$$M = M_F - \mu - \frac{\mu'}{2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (24)$$

Now the sum of the allocated moments is equal to

$$\frac{\mu'}{2} = \frac{K_{NO}}{2} \times \frac{\text{Joint moment}_N}{\Sigma K_{NO}} \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

The method is therefore self-checking by comparison of

$$\frac{\mu'}{2} = \frac{K_{NO}}{2} \times \frac{\text{Joint moment}_N}{\Sigma K_{NO}}$$

from Eq. 23*a* with half the complimentary term $\mu = K_{NO} \times \dfrac{\text{Joint moment}_N}{\Sigma K_{NO}}$ from Eq. 23*b*.

It should be noted that, since the rotation at a fixed end is zero, there is no allocated moment from a fixed end; hence, the entries in Cols. DG, EH, and FK are all zero.

TABLE 8.—Computation for Method of Moment Balance Using Fixing Moments

Quantity	Joint A			Joint B				Joint C		
	Member AD	Member AB	Σ	Member BA	Member BE	Member BC	Σ	Member CB	Member CF	Σ
K	3	5	8	5	4	5	14	5	3	8
(*a*) First Balance										
M_F	0	−8.4	−8.4	+2.8	0	0	+2.8	0	0	0
$\frac{\mu'}{2}$	−0.4	−1.2	−1.6	+2.6	+1.1	0	+3.7	−1.2	−0.8	−2.0
μ	+3.8	+6.2	+10.0	−2.3	−1.9	−2.3	−6.5	+1.2	+0.8	+2.0
M	+3.4	−3.4	....	+3.1	−0.8	−2.3	....	0.0	0.0	....
(*b*) Second Balance										
M_F	0	−8.4	−8.4	+2.8	0	0	+2.8	0	0	0
$\frac{\mu'}{2}$	−0.5	−1.4	−1.9	+3.1	+1.4	+0.6	+5.1	−1.4	−0.9	−2.3
μ	+3.9	+6.4	+10.3	−2.8	−2.3	−2.8	−7.9	+1.4	+0.9	+2.3
M	+3.4	−3.4	....	+3.1	−0.9	−2.2	....	0	0	....
(*c*) Third Balance										
M_F	0	−8.4	−8.4	+2.8	0	0	+2.8	0	0	0
$\frac{\mu'}{2}$	−0.5	−1.5	−2.0	+3.2	+1.5	+0.7	+5.4	−1.5	−0.9	−2.4
μ	+3.9	+6.5	+10.4	−2.9	−2.3	−2.9	−8.2	+1.5	+0.9	+2.4
M	+3.4	−3.4	....	+3.1	−0.8	−2.2	....	0	0	....

TABLE 8.—(*Continued*)

Quantity	Joint D				Joint E					Joint F			
	Member DA	Member DG	Member DE	Σ	Member ED	Member EB	Member EH	Member EF	Σ	Member FE	Member FC	Member FK	Σ
K	3	4	6	13	6	4	5	6	21	6	3	4	13
						(*a*) First Balance							
M_F	0	0	0	0	0	0	0	−11.5	−11.5	+5.2	0	0	+5.2
$\frac{\mu'}{2}$	$\underline{+1.6}$	0	$\underline{+1.6}$	+3.2	−0.7	−0.9	0	− 1.6	− 3.2	$\underline{+1.6}$	0	0	+1.6
μ	−0.7	−1.0	− 1.5	−3.2	+4.2	+2.8	+3.5	+ 4.2	+14.7	−3.1	−1.6	−2.1	−6.8
M	+0.9	−1.0	+0.1	...	+3.5	+1.9	+3.5	− 8.9	....	+3.7	−1.6	−2.1	...
						(*b*) Second Balance							
M_F	0	0	0	0	0	0	0	−11.5	−11.5	+5.2	0	0	+5.2
$\frac{\mu'}{2}$	+1.9	0	+2.1	+4.0	−0.9	−1.1	0	− 1.8	− 3.8	+2.1	+0.4	0	+2.5
μ	−0.9	−1.2	−1.9	−4.0	+4.4	+2.9	+3.6	+ 4.4	+15.3	−3.5	−1.8	−2.4	−7.7
M	+1.0	−1.2	+0.2	...	+3.5	+1.8	+3.6	− 8.9	....	+3.8	−1.4	−2.4	...
						(*c*) Third Balance							
M_F	0	0	0	0	0	0	0	−11.5	−11.5	+5.2	0	0	+5.2
$\frac{\mu'}{2}$	+1.9	0	+2.2	+4.1	−0.9	−1.2	0	− 1.8	− 3.9	+2.2	+0.4	...	+2.6
μ	−0.9	−1.3	−1.9	−4.1	+4.4	+2.9	+3.7	+ 4.4	+15.4	−3.6	−1.8	−2.4	−7.8
M	+1.0	−1.3	+0.3	...	+3.5	+1.7	+3.7	− 8.9	....	+3.8	−1.4	−2.4	...

Applying Eq. 24 to the moment balance system, the allocated moments $\left(\frac{\mu'}{2} = 2\,E\,K_{NO}\,\phi_N\right)$ are first estimated and added algebraically to the fixing moments to find the joint moment, which is then balanced. If, on comparison of $\mu = 4\,E\,K_{NO}\,\phi_N$ with $\frac{\mu'}{2} = 2\,E\,K_{NO}\,\phi_N$, the solution is not sufficiently accurate, the process is repeated, using one half of the last-found value of μ instead of $\frac{\mu'}{2}$ as the allocated moment in the next balance.

The compution is now as shown in Table 8, the order of balancing being joints B, D, F, A, C, and E and the initial estimates for $\frac{\mu'}{2}$ being obtained by balancing the fixed moment directly. Thus, +8.4 from joint A balances into AD = + 3.2 and AD = + 5.2. The initial estimates are underlined in Table 8. Values from the revised estimates have been used as soon as found.

It will be seen that, although the second balance is quite close enough for practical purposes, the calculation has been taken to the third balance as a check. It also will be noted that the values in the moment balance (Table 8)

for $\frac{\mu'}{2}$ and Σ are in all cases the same as the sum of the figures at the corresponding stages in the method of allocated moments (Table 7).

It should be pointed out that the example taken is a very simple one, the incidence of the loading being such that the allocated moments in any one column are always of the same sign and that the joint moment, therefore, is always increasing in absolute value. This would not necessarily be the case if all the bays were loaded.

The outstanding merit of the method suggested by Mr. Cornish is that it enables a very rapid first approximation to be made of the moments at a joint in terms of the members immediately adjacent to that joint. It is of great value in the initial stages of a design, and perhaps at a time when the other members have yet to be designed, to be able to arrive at this first approximation. Any such approximation can also be checked and corrected, as the design of the members proceeds, by taking out subsequent balances.

It is submitted, however, that the introduction of the concept "*X*" is unnecessary.

Although it is admitted that members remote from the joint under consideration only affect the moments at that joint in a secondary degree, clearly any method which achieves quickness by discarding certain factors must be limited in that respect.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EARTH PRESSURE AND SHEARING RESISTANCE OF PLASTIC CLAY

A SYMPOSIUM

Discussion

BY A. A. EREMIN, ASSOC. M. AM. SOC. C. E.

A. A. EREMIN,[27] ASSOC. M. AM. SOC. C. E.[27a]—In the computation of the soil pressures on the tunnel lining, Professor Terzaghi correctly has stated (see heading "I.—Clay Pressure on Temporary and on Permanent Tunnel Support: Clay Pressure on the Finished Tubes") that "It would be idle to attempt to answer this question on purely theoretical grounds." However, any one making an experimental study of the various factors affecting the distribution of the tunnel pressures cannot attain a reliable result without a rational basic theory. Furthermore, in order that the method of computation may be applied to the various cases of tunnels and tunnel loadings, the assumptions must have the least possible number of limiting conditions.

In Fig. 3(*b*) Professor Terzaghi assumes that the width of the soil prism that produces a load on the tunnel lining is equal to the sum of the tunnel width and twice the height of the tunnel walls. Generally, the height of the tunnel walls varies. In highway tunnels, the entire tunnel section often is formed by the arched section. Evidently, in tunnels with low walls, Professor Terzaghi's method of constructing the soil prism will not represent the characteristic form for the pressures on a tunnel lining. In Eq. 2 Professor Terzaghi assumes that the vertical load carried by the soil at the back of tunnel walls is equal to $2\ q_r\ H_1$. It is to be noted that this is true only in the case of failure or when the tunnel footings are yielding. Obviously, the footing pressures shown in Fig. 9 at the initial step of failure will be much greater than that shown by Professor Terzaghi.

In his paper Professor Housel has developed the "circuit method" for computing the pressures in the tunnel lining, based on various approximations.

NOTE.—This Symposium was published in June, 1942, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: September, 1942, by Messrs. Ralph H. Burke, L. G. Lenhardt, George E. Shafer, and M. E. Chamberlain.

[27] Associate Bridge Engr., Bridge Dept., Div. of Highways, State Dept. of Public Works, Sacramento, Calif.

[27a] Received by the Secretary November 16, 1942.

He states (see heading "Analysis of Earth-Pressure Measurements") that "The validity of such approximations is difficult to prove or disprove except by the final test of whether or not they reproduce the measured pressures." True, it is difficult to prove the relation between Eq. 32 and Eq. 34*a*. In Eq. 32 the shear factor expressing the horizontal pressure on the tunnel lining as transformed from the vertical pressures in the earth prism is 4 S. In Eq. 34*a*, on the other hand, the shear factor expressing the upward vertical pressure on the invert arch in the tunnel lining as transformed also from the vertical pressures in the earth prism (or with respect to a change in direction of stresses by an angle of 180°) is only 5 S.

Eq. 34*b* is developed for use in checking the pressures on the invert arch of the tunnel lining. Since friction on the sides of the circular tunnel lining is negligible, Eq. 34*b* could be considered as an approximate equation for the computation of the pressures on the inverted arch, whereas Eq. 34*a* is more suitable for checking the assumed shear factors.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

RELATION OF UNDISTURBED SAMPLING TO LABORATORY TESTING

Discussion

BY MESSRS. BENJAMIN K. HOUGH, JR., AND F. M. VAN AUKEN

BENJAMIN K. HOUGH, JR.,[3] ASSOC. M. AM. SOC. C. E.[3a]—Although the paper is entitled "Relation of Undisturbed Sampling to Laboratory Testing," the author has extended his remarks to include a discussion of the broader subject of the practical application of test results in soils and foundation engineering problems. This more comprehensive view of the subject is justifiable since soil sampling and testing are obviously only initial steps in solving practical soil problems and no matter how well they may be performed they cannot be considered an end in themselves. Furthermore, it can be maintained that the degree of precision which is necessary in sampling and testing depends on the nature of the analytical method in which the data are to be used. On this basis, it is possible that in some cases more time and money have been expended in obtaining undisturbed samples than can be justified in view of limitations in the method of applying the test data. For example, expense is often a major consideration in deciding whether to conduct exploration in test pits or by drilling. The writer does not entirely agree with the view that test-pit samples are superior to those from drill holes; but even if this were true, it would not follow that test pits are always preferable unless it is also true that they are, in general, more economical. It is the writer's experience that test pits more than about 5 ft deep, and particularly test pits that are extended below ground water, are considerably more expensive than even fairly elaborate drilling operations and, for this reason, test pits should not be used unless the results obtained clearly justify the additional expense.

In discussing ways of determining whether soil samples have been disturbed and, if so, to what extent, Professor Rutledge refers to data from consolidation and shearing tests on the best obtainable undisturbed samples and on the same material remolded. One of the materials referred to as being practically undisturbed is a volcanic clay from Mexico City which was obtained by manual

NOTE.—This paper by P. C. Rutledge, Assoc. M. Am. Soc. C. E., was published in November, 1942, *Proceedings*.

[3] Capt., Corps of Engrs., U. S. Army, U. S. Engr. Office, Boston, Mass.

[3a] Received by the Secretary November 24, 1942.

sampling in an open pit. A characteristic of this clay is that it contains more than 90% voids. It is assumed by the writer that in addition to the care which was exercised in the sampling operation, some very special precautions were taken in packing and shipping this rather delicate material to the Harvard testing laboratory since considerable disturbance might well be caused by rough handling during transportation. In view of the real grounds for suspecting that some injury might have taken place during shipment, the writer feels that it might be advisable to omit this reference or to substitute data from tests made in the locality from which the material was obtained.

A further comment on the author's treatment of consolidation tests is that he has possibly created the impression that if a sample is practically undisturbed the compression-test diagram will have a certain more or less standard character such as that illustrated in Fig. 2. Two features of this idealized diagram, which are not always found in actual practice, are the straight-line "natural-soil" compression curve and the complete return to this curve of each cycle of reloading. That the "undisturbed-soil" curve is not always straight is evidenced by the author's tests on the Mexico City clay, and also by tests recently made by the writer on certain St. Lawrence River clays with an initial void ratio of about 1.6 instead of the unusual value of 14.0. Definite curvature in practically all the semi-log compression diagrams for these Laurentian clays was found to be characteristic. It has also been observed by the writer that, more often than not, release of load and subsequent recompression during a consolidation test has the effect of offsetting the compression diagram somewhat to the left so that the successive cycles fail to form an extension of the original or undisturbed-soil curve. It is not the writer's belief that these discrepancies between actual test curves and the idealized diagram in Fig. 2 are evidences of undue disturbance of the test specimens during sampling operations. The writer agrees, however, that sample disturbance does obscure the preconsolidation point and that great care in sampling must be exercised when a determination of this point is a matter of importance.

In connection with the Terzaghi theory to which the author refers, that the amount of compression produced by application of a given load may be influenced by the rate at which the load is applied, the writer presents some data obtained during tests on the previously mentioned St. Lawrence River clays. At one point where subsurface investigations were made by the writer, the clay deposit is approximately 60 ft deep. The upper 10 ft or 15 ft of this deposit is altered somewhat by oxidation and has dried out to some extent. In the remaining depth, however, the material is remarkably uniform in appearance and even lacks the usual stratification and traces of bedding planes which mark many clay deposits. In this uniform section of the deposit, water-content determinations were made on samples taken at fairly frequent intervals, and it was found that the water content (and hence the void ratio) of the material remained substantially constant with depth. In fact, in one location there was a slight trend toward higher water contents near the bottom of the deposit than at the top. Water contents in all cases, incidentally, even at 60-ft depths, were found to be well above the liquid limit, a fact which is in contrast with the usual assumption that clays are laid down initially at the

liquid limit. This clay is classified as a post glacial, marine deposit which has never been subjected to any loading other than its own weight and its age is sufficient to warrant the assumption that the clay is fully consolidated under its own weight. It would therefore appear reasonable to expect that samples taken from the bottom of the deposit would be in a more dense condition than those at the top instead of having practically uniform density as indicated by the data from water content determinations. This apparent discrepancy may be explained by the theory that rate of loading is a very important consideration and that the "sedimentation curve" may be nearly flat instead of an extension of the relatively steep compression curve obtained from laboratory tests. It also has bearing on certain aspects of the shearing strength of clays, a subject which the author discusses in connection with the effects of sample disturbance.

From the aforementioned evidence it appears to be a physical possibility for a clay deposit of considerable depth to exist without variation in void ratio from top to bottom. Despite this fact, it seems undeniable that intergranular stresses must increase steadily with depth in such a deposit, provided it is fully consolidated. Under these circumstances, it might well be expected that the shearing strength of the clay would gradually increase with depth in proportion to the increase of intergranular pressure; yet in the case of the St. Lawrence clay the evidence from drilling records, from visual inspection of the material in place in open test pits, and from an extensive series of direct, unconfined, and triaxial shear tests is that there is no appreciable change in shearing strength with depth. This finding, if valid, is a direct contradiction to the widely accepted Coulomb relation for the shearing strength of soils and establishes grounds for a major revision in many current soil theories. Space does not permit extensive treatment of this subject, but it is interesting to consider the possibility that at least in clay deposits of similar origin, shearing strength is to some extent a function of void ratio and hence water content, rather than being entirely due to intergranular pressure. Laboratory experimentation on this subject would be a difficult matter for, although nature can apparently produce deposits with uniform void ratio yet increasing intergranular pressure, any change of pressure in laboratory samples results in a change of void ratio. If shearing strength is suspected of varying with either void ratio or pressure, it is difficult to determine the basic relationship in a test in which both these factors vary.

To turn briefly to the author's suggested methods of applying test data, the writer agrees that purely theoretical methods are unsatisfactory and also that adherence to purely empirical methods is not always wise; but, of the two, the writer is inclined to favor the latter. The method designated by the author as semi-theoretical might perhaps be termed the rational method. Whatever name is used, however, for analytical methods which combine experience, judgment, and engineering ability, the writer feels that emphasis should be given to the need for careful substantiation of each new development before it is expounded. On the whole, the writer agrees that recent studies in soils engineering have given better insight into the behavior of soils than hundreds of years of empiricism, but it is also true that the path of progress has been

considerably cluttered with theories which have been hastily proposed and, with equal or greater haste, discarded.

In closing, the writer suggests that the author's selection of the Terzaghi approximate relationships as an example of the preferred "semi-theoretical" method of applying test data is unfortunate. It is the writer's understanding that these approximations were developed for idealized soil types seldom if ever encountered in actual practice. Furthermore, the writer has found that the Terzaghi approximate relationships are not in agreement with the observed behavior of large-scale field tests, specifically tests to determine the effect of the size of loaded area on the bearing power of cohesionless soils.

F. M. VAN AUKEN,[4] ESQ.[4a]—An excellent treatment of the relation of undisturbed sampling to laboratory testing has been presented in this paper. The methods that can be used as "pilot" tests to determine whether or how much a sample is disturbed are particularly enlightening.

Engineers have long recognized that laboratory tests performed on disturbed and partly disturbed soils are not in themselves of great value. Fair interpretation of results is difficult; consequently, the data are usually relegated to the "files" where they well belong.

It is fortunate for the soils engineer that only two major test divisions require the use of undisturbed soil specimens. Once the consolidation and strength characteristics of a soil are known completely, all other tests become mere pawns of the former. However, since the scope of the present paper is limited entirely to plastic soils—soils that under stress generally disclose great volume changes—its scope is somewhat narrow and cannot hope to cover, even by indication, the entire field of undisturbed sampling and testing. Unfortunately this is not the case. The choice of highly compressible clays as the subject for discussion, although they do furnish unusually fine examples for a study of this type, may indicate that it is relatively easy to determine the state of disturbance that invariably accompanies all attempts to sample soils.

Probably no one consideration is of more importance than a complete reconstruction of the geologic history of the soil before it is sampled and tested. In fact, a straightforward dependence on the so-called "preconsolidation" load, as determined by the consolidation test without a thorough knowledge of past geologic history, is probably as bad as not knowing whether a given material is disturbed.

Certain clays, such as the volcanic clays from the bed of Lake Texcoco, Mexico, D. F., for which the geologic history is well known and which are compacted to abnormally high void ratios, make perfect examples of the determination of the preconsolidated load because of the "classic, almost photographic enlargement," of the consolidation test features.

Other clays like the glacial clays from the Great Lakes region, in which the geologic history is more obscure, are difficult to rate with respect to determination of the preconsolidation load and condition of soil. Recently reworked alluvial, plastic clays also present problems identifying sample disturbance.

[4] Chf. of Soils Laboratory, U. S. Engr. Office, Denison, Tex.

[4a] Received by the Secretary November 30, 1942.

Other soils that present even more difficulty in the determination of condition are marly and shaley clays. Although they are somewhat brittle in their initial state of consolidation, these materials become truly plastic when the structure is broken down and the material remolded. Soils of this type are usually laid down as sedimentary deposits and consolidation effected by the weight of material that the forces of nature have long since removed. It may be probable that periods of recompression have occurred, caused by deposition of additional material, with re-expansion upon its removal, so that the number of compression and expansion cycles to which the material has been subjected is problematic. One may never know whether a soil is now in a period of compression or expansion. Even with an accurate knowledge of that fact the determination of the preconsolidation load by means of the shape of the pressure, void-ratio curve is practically impossible. Additional complications arise when primary expansion occurs.

Although the effect of disturbance is pronounced on the determination of the consolidation characteristics of a soil, it is not a factor that will vitally affect a consolidation study which (because of the number of theoretical assumptions, drainage, and boundary conditions, etc.) is in itself only a fair estimate of expected settlement. On the other hand, the effect of disturbance on the strength of soils is profound. Soils under vertical stress undergo a gradual rearrangement of internal structure as the stress is slowly increased, thereby resulting in consolidation or reduction in volume. Assuming a somewhat flexible confining restraint such as would be afforded a loaded element in a soil mass, the investigator will note that, as the vertical stress is increased, although consolidation is occurring, a gradual approach to a state of shear is apparent. Hence, a semiconfined stressed soil is only in a transitory stage which, as pressure is increased, rapidly progresses into fully developed shear, and when no further strain can be taken either ruptures suddenly or undergoes metamorphosis and fails in plastic flow. Disturbance in sampling merely places a soil well along in that vague but nevertheless always present stage in which the application of even a minor stress could produce failure. Therefore, regardless of testing technique, tests that are used to measure the strength of soils should show a corresponding percentage decrease in strength if a material is disturbed during or after sampling.

The writer concurs with Professor Rutledge in passing over the direct shear test rapidly, not because the effect of disturbance cannot be ascertained, but because the test in itself furnishes data that are difficult to apply. Although the slope and shape of the stress-strain curve may furnish data from which possible disturbance of a soil specimen may be detected, it is quite probable that this is true only for sensibly plastic materials. Unfortunately the ordinary range of soils runs the gamut of plastic to brittle materials; hence, it may be that internal grain structure, pseudo structures set up by fusion or cyclic desiccation, relation of moisture content to capillary and chemical forces, have as much to do with the determination of the apparent shape of the stress-strain curve as possible disturbance. The simple compression test is probably a better test than the direct shear test, but since the application of a primary

vertical stress sets up secondary stresses over which the investigator has no control it appears that the triaxial test would have more significance.

Professor Rutledge has condemned the triaxial test for the simple reason that some specimens tend to consolidate during the test, and since disturbance accentuates consolidation, as shown by consolidation test research, unmeasurable errors may be introduced. The writer takes exception to this theory on the basis that the approach to the shear problem absolutely necessitates volume changes. In a coplanar system, any loaded mass of soil will be subjected to major and minor stresses which, for given boundary conditions, have some definite ratio. To alleviate the effect of preconsolidating a test specimen prior to application of deviator stresses, there are a number of methods of doing the triaxial compression test. One such method is to set up a particular stress ratio $\frac{\sigma_1}{\sigma_3}$ and maintain that ratio until the volume of the soil is stabilized; then increase the major principal stress until failure occurs. In this manner the investigator has attempted to simulate what may happen in nature.

Professor Rutledge has shown a plot of stress-strain curves for the so-called quick and slow triaxial compression tests, but it is pointed out that no effect of disturbance can be ascertained from a singular comparison of these tests. In addition, the final strength obtained from the slow test is a function of time; that is, one can obtain increasing strength values depending on whether the strain is applied at the rate of 1% per minute, per hour, per day, or per month. This fact has been substantiated by slow triaxial compression tests performed in the writer's laboratory in conjunction with the design of a dam where some tests were continued nearly 12 months before failures were obtained. Tests on remolded specimens, as in the case of the simple compression tests, disclosed far lower ultimate strengths and flatter stress-strain slopes.

If it is assumed that a perfectly undisturbed sample retains more elasticity than a disturbed specimen (that is, the structure set up by nature has not been sensibly altered), one can determine with a certain degree of success the effect of disturbance by cyclic loading, provided the strain has not exceeded 25% to 50% of the ultimate. Other triaxial compression-test series, performed in the writer's laboratory, have shown that the apparent preconsolidation load can be determined approximately by the proper combination of quick and slow tests, provided the samples were truly undisturbed.

A review of the various proposed methods of determining how much disturbance affects soil test results shows that all methods are still problematic in aspect. That disturbance does affect both consolidation and strength values is a fact that cannot be overlooked. However, during any complete soil investigation, the scope of the problem is often such that only pilot tests to determine how much disturbance is being caused by the adopted sampling methods can be performed. On the basis of those tests one must either discard or improve the sampling technique or recognize the limitations of the soil test data obtained. In years past, soils engineers, although probably recognizing the effect of sample disturbance on test data, were prone to pick the "yield point" from the stress-strain curve, resulting in the use of values far below the

ultimate ones. However, during the past few years, ultimate values are more generally used, but the one redeeming feature that has covered many errors is the fact that sample disturbance generally lowers the obtained values.

Professor Rutledge's paper will have accomplished its prime purpose if only through stimulation of a timely subject. In closing, the writer wishes to emphasize two facts often overlooked and having a decided effect on the accuracy of soil-test data. Those facts are "too long" a time lapse between sampling and testing, and "too little" emphasis on the choice of sampling and testing technique.

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

CONFORMITY BETWEEN MODEL AND PROTOTYPE

A SYMPOSIUM

Discussion

By A. E. Niederhoff, Assoc. M. Am. Soc. C. E.

A. E. Niederhoff,[15] Assoc. M. Am. Soc. C. E.[15a]—About the only thing that can be added to the thorough and fine paper by Messrs. Nelson and Hartigan is to call attention to corroborative evidence found on the West Coast. This evidence concerns two high lift locks with lifts of 66 ft and 45 ft, respectively.

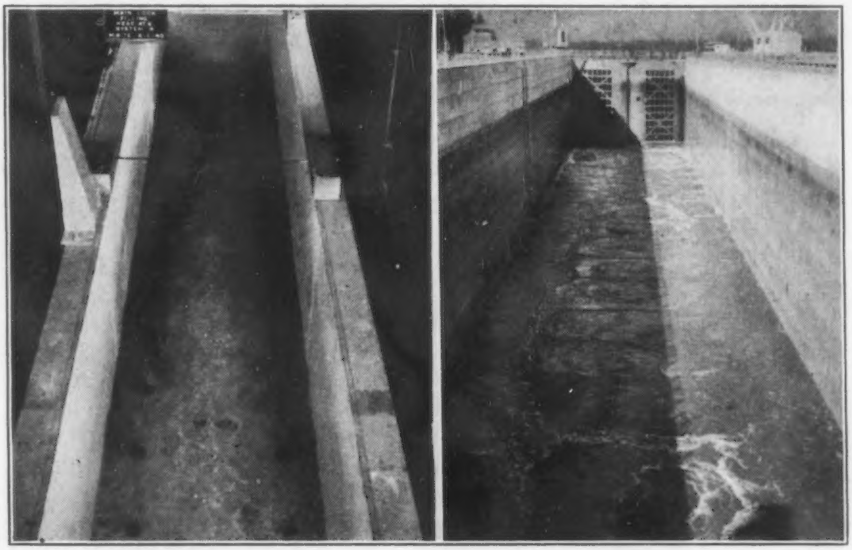
The authors have revealed the fact that, where geometric similarity does not exist between model and prototype, correlation of test results is difficult. Of this there can be no doubt but, since total similarity, geometric and dynamic, can never be obtained when the same liquid is used in both model and prototype, one should not be too disturbed at failure to obtain exact quantitative results. After all, a model of any kind is not the answer to all questions on a subject. At the risk of seeming irreverent, the writer calls attention to the model husband who has been defined as "a small imitation of the real thing." Hydraulic models fall in the same category, and engineers do well to observe trends and characteristics in a model that are duplicated in the prototype. Actual measurements of time, velocity, forces, or discharges are done best on a full-size lock for possible design use on a similar structure.

Fig. 62(*a*) shows the filling of a model lock, utilizing a floor culvert and port system patterned after one of the locks discussed herein (66-ft lift). The same water disturbance in the model chamber will be observed in the prototype (see Fig. 62(*b*)) especially at the far end, near the lower miter gates. Also, when emptying the model chamber, the fountain effect below the lower miter gates is duplicated every time the 66-ft lock is emptied. It is interesting to note that, in spite of lack of exact similarity, the coefficient of filling for

Note.—This Symposium was published in October, 1942, *Proceedings*.
[15] Senior Structural Engr., U. S. Engr. Dept.
[15a] Received by the Secretary October 22, 1942.

the model and prototype agreed within 1%. An average coefficient of filling was 0.83, and, for emptying, this coefficient dropped to 0.72.

Hawser stresses in model lock chambers have been measured as a criterion for proper functioning for several years. However, to the writer's knowledge, only one attempt to measure hawser stresses has been made on a high lift lock. In 1940 a government derrick boat was moored by four lines to floating mooring bits in the 66-ft lock. Dynamometers were placed in these lines and



(*a*) Model (Head, 47.6 Ft) (*b*) Prototype (Head, 49.0 Ft)

Fig. 62.—Disturbed Water Condition When Lock Chamber Is Being Filled

readings were taken simultaneously at each 5 ft of lift. When plotted as a curve, the results showed that the maximum pull occurred when the valves were fully open and when discharge into the lock was a maximum. The curves showed the same family characteristics as those obtained in a model even if the actual magnitude of the pull on the lines was considerably greater than that shown in the model.

The distribution of water from ports in a lock chamber has been ably described by Messrs. Nelson and Hartigan, and it is only in deference to Southern California that the writer wishes to point to the full-scale results of the "manifold problem" investigated at the second high lock mentioned herein (45-ft lift). Southern California was a factor in this test because the grapefruit used as floaters came from Coachella Valley. These juicy, yellow spheres were placed just upstream from the filling valves, and timed and counted as they emerged from the several chamber ports. In this particular case, most of the grapefruit, and therefore most of the water, came from the port farthest upstream in the chamber.

Model tests have shown the necessity for relieving entrapped air in a culvert system if the lock is to be filled efficiently. The conditions at the 45-ft lock at the time of conversion from a tandem lock to a single lift of 45 ft were such that only one large vent hole per culvert could reasonably be drilled. This hole was 20 in. in diameter. When completed, it worked beautifully, sending a geyser of air and water 60 ft into the air above the top of the lock-wall and adding another spectacle to the scenic beauty of the region.

When the diameter of the vent hole was reduced to 10 in. by placing a pierced diaphragm over the end of the vent pipe, the water no longer shot out of the vent pipe but entrapped air exploded into the lock chamber. Cutting the effective vent hole to a diameter of only 4 in. merely made the chamber explosions more violent and more prolonged.

Correction for *Transactions:* In October, 1942, *Proceedings*, page 1268, correct line 10 as follows: "Lieutenant-Colonel Thompson," and "Colonel Vogel."

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PERMEABILITY OF MUD MOUNTAIN DAM CORE MATERIAL

Discussion

BY F. H. KELLOGG, M. AM. SOC. C. E.

F. H. KELLOGG,[6] M. AM. SOC. C. E.[6a]—The fact that distribution of voids in a soil, rather than total voids volume, may be a diagnostic criterion of soil behavior has been excellently and strikingly demonstrated by the authors. The results of a number of permeability tests on several soil types are shown, in Fig. 7, to give results similar to those shown in Figs. 5 and 6. Such behavior is of practical significance in several ways, particularly in considering the swelling and softening of a compacted soil due to wetting.

Experience with fine soils compacted to form very hard, resistant masses, at moistures lower than optimum, has indicated that subsequent wetting of these masses results in a striking loss in strength. The wetting frequently causes considerable swelling. The same soils, compacted at moistures higher than optimum, show little or no swelling and softening after subsequent wetting. This has led to the control of moisture for such purposes as minimizing swelling of a highway subgrade under a concrete slab.[7] These effects of compacting moisture on swelling and softening, as actually observed, could be predicted, qualitatively at least, by test results such as those shown in Figs. 8 and 9. Fig. 8 shows, as points I, II, III, etc., the moistures and densities of a soil as compacted, and, as points 1, 2, 3, etc., the corresponding moistures and densities after wetting subsequent to compaction. Fig. 9(*a*) shows, as ordinates, the coefficients of internal friction of a soil that was submerged and saturated after being compacted at the moistures shown as abscissas; and Fig. 9(*b*) shows corresponding data for cohesion. These data, together with those presented by the authors, indicate the disadvantages of compacting fine soils at less than optimum moisture, due to the comparative ease with which water can find its way between hard lumps in the soil, softening them and weakening the entire mass. This was graphically illustrated by experiences with subgrades of A-5-7-type soils,[8] where time requirements and availability

NOTE.—This paper by Allen S. Cary, Assoc. M. Am. Soc. C. E., and Boyd H. Walter, and Howard T. Harstad, Juniors, Am. Soc. C. E., was published in September, 1942, *Proceedings*.

[6] Senior Materials Engr., TVA, Knoxville, Tenn.

[6a] Received by the Secretary October 16, 1942.

[7] *Engineering News-Record*, March 18, 1937, pp. 405–409.

[8] *Public Roads 12*, July, 1931, pp. 136–137.

of materials prevented selection or special treatment of subgrade materials. Subgrades were either opened to traffic compaction or compacted at controlled moisture just before placing the 6-in. mechanically stabilized base and the asphaltic seal courses. After 2 to 4 months of predominately rainy weather, subgrade failures occurred at several places. At the points of failure, the subgrade had become wet and highly plastic. After checking for the effects

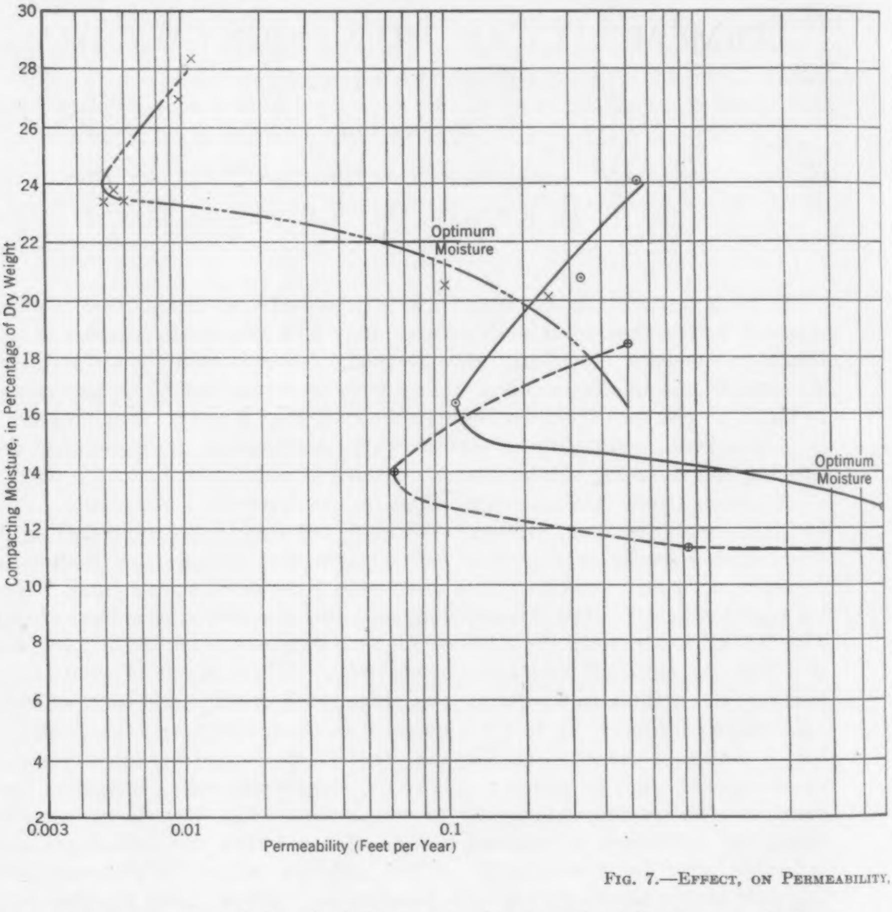


FIG. 7.—EFFECT, ON PERMEABILITY,

of coarse aggregate and chemical changes in the clay, the following conclusions were made:

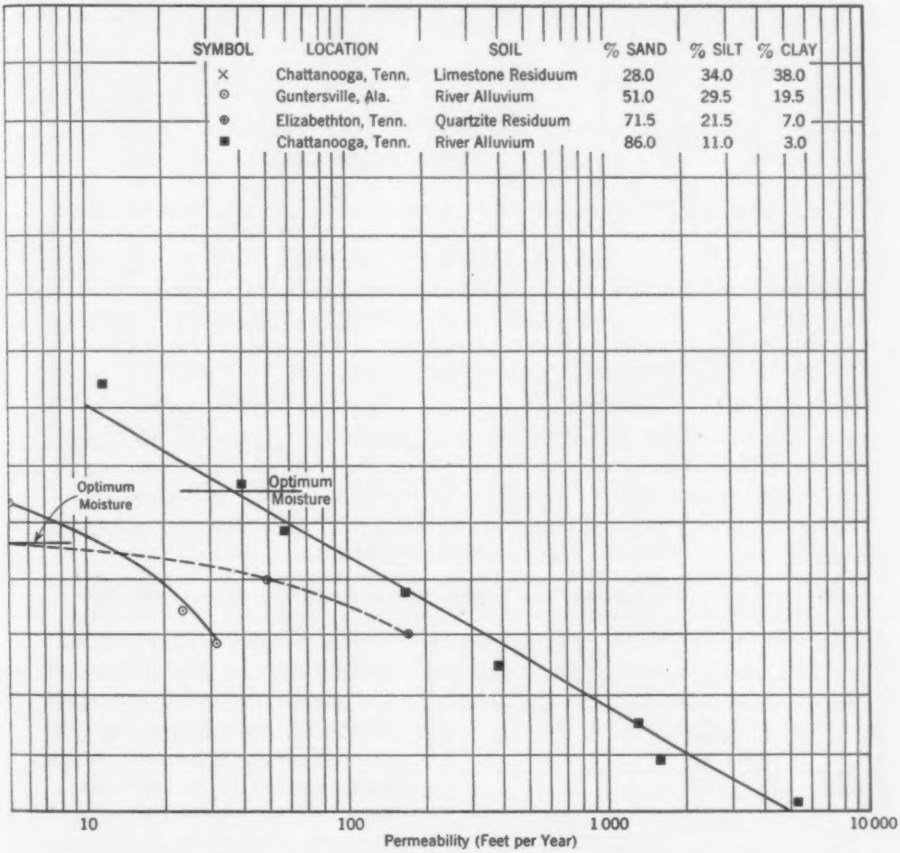
(1) Smooth, hard subgrades compacted at moistures lower than optimum were softened during wet weather to a plastic clay incapable of supporting the required wheel loads. This was true in spite of measures taken to promote drainage.

(2) Subgrades compacted at moistures higher than optimum and immediately covered by the slab would not soften appreciably.

(3) Compacted clay subgrade could not be successfully plowed, moistened,

and recompacted just prior to paving, as insufficient time for uniform distribution of moisture was available. Such a procedure merely served to wet the surface of the clay lumps, without affecting the compacted mass as a whole.

In the construction of certain impounding dikes, the relationships demonstrated by the authors have been used to advantage where only one type of material was available. At the upstream face of such a dike, high compacting



of Compacting Moistures

moistures were used, giving a fill of low permeability. Downstream, lower compacting moistures gave a more pervious fill, increasing the effectiveness of the drains and partly offsetting the effects of anisotropy. Such a procedure, of course, must be consistent with the stability requirements of the dike, and the upstream fill must not be compacted when it is so wet as to cause cracking under wheel loads.

In the writer's experience, any soil that can be compacted at a given moisture to within 4 lb of the density which would permit no air voids at that moisture, and that has a soil mortar with an appreciable plasticity index, will deviate from the "normal" permeability-void ratio relationships. In other

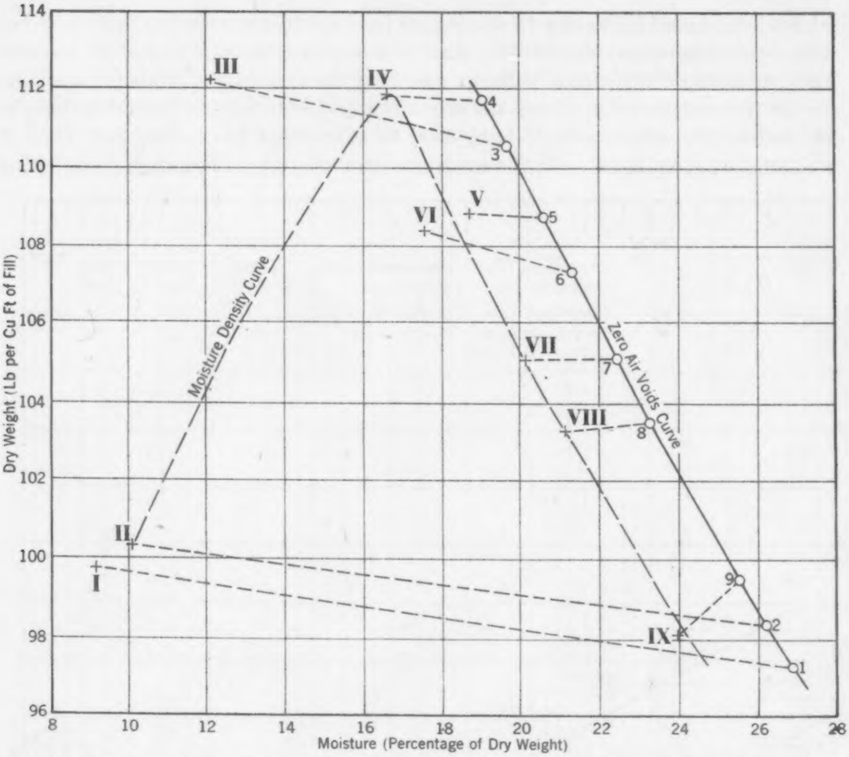
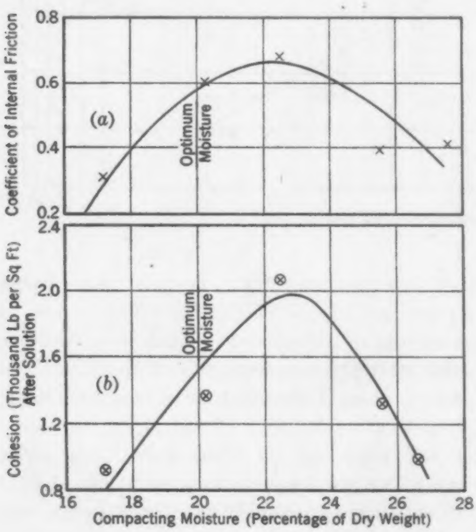


FIG. 8.—EFFECT OF COMPACTION ON VOLUME CHANGES DUE TO SUBSEQUENT WETTING UNDER LOAD OF 160 LB PER SQ FT (SOIL A-5 SILT LOAM, TENNESSEE RIVER FLOOD PLAIN, CHATTANOOGA, TENN.)



words, materials containing sufficient clay to form lumps at low moistures, and to lubricate coarse grains at high moistures, should fall into this class. Since montmorillonite is particularly sensitive to physicochemical changes, materials containing appreciable quantities of that mineral would require a lower percentage of fines to bring them into this class than would other materials.

FIG. 9.—EFFECT OF COMPACTING MOISTURES ON FRICTION AND COHESION OF SATURATED SUBMERGED SOIL (SOIL A-5-7 IS A CLAY LOAM, WITH DOLOMITE RESIDUUM, FROM DANDRIDGE, TENN.)

AMERICAN SOCIETY OF CIVIL ENGINEERS
Founded November 5, 1852
DISCUSSIONS

STATISTICAL ANALYSIS IN HYDROLOGY

Discussion

BY R. H. BLYTHE, JR., ESQ.

R. H. BLYTHE, JR.,[27] ESQ.[27a]—Failure of practical application to meet conditions that are the fundamental bases of the several mathematical theories of flood-frequency curves makes mathematical refinement in the fitting of such curves a meaningless labor. Furthermore, flood-frequency curves find their most justifiable use in the solution of economic problems, such as the estimation of average annual damage, rather than in the forecasting of the maximum flood for design purposes. For these economic problems, the lower and better-defined parts of the curves are usually at least as important as the extremities, which show the frequency of the larger floods. Extrapolation of the upper end of such a curve is certainly an unjustifiable procedure in view of the discrepancy between natural conditions and the assumptions underlying the theories.

Since these things are so, a curve fitted freehand to plotted points would seem to be the logical method of expressing the flood-frequency information contained in the usual record of flood flows; but at this point the question of the plotting position of the points arises. Currently, there seem to be two major opinions on this subject, and a number of minor ones, including the hypothesis presented in the paper.

To put the problem simply and concretely, consider a series of twenty occurrences. It makes no difference whether they are maximum annual floods, the twenty largest floods of record, or some other similar series. These occurrences are assumed to represent a small sample of an indefinitely large population. The mean of this sample can be considered an estimate of the mean of the population of events specified in the same way as the sample. Also, assume that the maximum observed event in the sample affords some information about the largest 5% of the population. In this sample a value

NOTE.—This paper by L. R. Beard, Jun. Am. Soc. C. E., was published in September, 1942, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: October, 1942, by Messrs. L. Standish Hall, and H. Alden Foster; and November, 1942, by Messrs. Paul V. Hodges, and Joe W. Johnson.

[27] U. S. Dept. of Agriculture, Forest Service, Washington, D. C.

[27a] Received by the Secretary November 12, 1942.

equal to this maximum has occurred with a frequency of 5%. Therefore, the supporters of the "California method"[28] (a very simple and ordinary concept to be called a "method") contend that the maximum flood should be plotted as having a frequency of 5%; but Mr. Foster and his followers[16] point out that there is no reasonable basis for assuming that this one particular maximum event in a sample of twenty represents the exact 5% frequency point. They contend, rather, that it represents the entire class of events occurring with a frequency of 5% and less. They state that, since it represents the interval 5% to 0, a reasonable point for plotting is the midpoint of this frequency interval, or 2½%. This seems more logical than the California technique. Statistically, the assumption is that the one maximum occurrence observable in a series of twenty is the median value of the twenty values that would compose the upper 5% of a sample of 400, or the median value of the upper 5% of the infinite population it is taken to represent.

There seems to be still more logic in assuming that the maximum observed occurrence in one sample of twenty is the mode of the distribution of extreme values in a number of samples of twenty observations. This follows from the fact that the mode, by definition, occurs most frequently, and therefore any single value chosen from a non-normal distribution is more likely to be the mode than the average or the median. In any normal distribution the three coincide, but the distribution of the extreme values of samples drawn from a normal population is non-normal.

Studies of the distribution of the extreme values of samples of n observations taken from an infinite normal population have been made by L. H. C. Tippett,[29] who gives tables of the distribution of the extreme values of samples of several sizes from $n = 3$ to $n = 1{,}000$. The values of the variate are given in terms of the standard deviation of the original population; also given are tables showing the mean, standard deviation, and β_1 and β_2 (coefficients of skewness and kurtosis) of the distributions for $n = 2$ to $n = 1{,}000$. From these tables it was possible to compute the value of the mode by the formula:

$$\text{Mode} = \bar{x} - \sigma \frac{\sqrt{\beta_1}\,(\beta_2 + 3)}{2\,(5\,\beta_2 - 6\,\beta_1 - 9)} \qquad (5)$$

in which $\bar{x}$ = the mean, and σ = the standard deviation. Values of the mode thus computed are shown in Table 5. It should be noted that they are expressed in terms of multiples of the standard deviation.

If these are the most likely values for the extreme observation of a single sample, then the probability corresponding to this modal value is the most likely estimate of the probability (or frequency) of the extreme observation of the single sample. The probabilities corresponding to the modal values for samples of different sizes are given in Col. 3, Table 5. They were found by looking up the modal values in tables of the normal distribution.

---

[28] "Floods in the United States," *Water Supply Paper No. 771*, U. S. Geological Survey, 1936.

[16] *Ibid.* (Chapter on Methods for Estimating Floods, by H. Alden Foster).

[29] "On the Extreme Individuals and the Range of Samples Taken from a Normal Population," by L. H. C. Tippett, *Biometrika*, Vol. XVII, Pts. III and IV, p. 364.

By this hypothesis, the tabulated probabilities are the plotting positions of the extreme value of a sample. Also listed in Table 5 (Col. 4) are the plotting positions according to Mr. Foster's hypothesis. It will be noted that the differences are not large. It would seem logical for practical purposes to adopt his method as giving a reasonable and simple approximation of the best estimate. Because of the failure of actual flood occurrences to form a normal, distribution, and because of other discrepancies between theory and practice the labor involved in calculating the mode and its probability is unjustifiable. Of course, Table 5 could be used, and values for other sizes of sample could be interpolated.

TABLE 5.—MODE, PROBABILITY, AND PLOTTING POSITION OF EXTREME VALUE IN SAMPLES OF DIFFERENT SIZES

Size of sample (n) (1)	Mode of extreme values (2)	Probability (3)	Plotting position (4)
2	0.50851	0.306	0.250
5	1.06553	0.143	0.100
10	1.42421	0.077	0.050
20	1.74305	0.041	0.025
60	2.18877	0.014	0.0083
100	2.37639	0.0087	0.005
200	2.60510	0.0046	0.0025
500	2.90778	0.0018	0.0010
1,000	3.11482	0.00092	0.0005

Unfortunately, the problem of where to plot the next highest and succeeding floods remains to be solved. This is principally a matter of theoretical interest and is beyond the mathematical abilities and patience of the writer.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EARLY CONTRIBUTIONS TO MISSISSIPPI RIVER HYDROLOGY

Discussion

By JOHN C. HOYT, M. AM. SOC. C. E.

JOHN C. HOYT,[28] M. AM. SOC. C. E.[28a]—The pioneer work in the collection of river flow by Humphreys and Abbot[2] and others on the Mississippi River indicated a fair recognition of the fundamental principles of the subject. The principles then recognized have stood the tests of later studies of the subject, and they have been the basis for the further development of the science of river hydrology.

Mr. Jarvis' paper brings together the results of much of the early work on the Mississippi River in a form that makes them readily available for further comparison and study. The comparison of records of river discharge taken at different times depends on three main factors: (1) The methods used in their collection; (2) the equipment and facilities available for collecting the records; and (3) the permanency of conditions along the streams which regulate the flow.

Although the methods of measurement are fundamentally the same, the equipment and facilities for making the measurements have been greatly modified and improved so that the accuracy in recent work has been greatly increased; for example, improvement in sounding equipment has added materially to the accuracy of records. Whereas some streams have fairly permanent channels which govern their regimen, many of the larger ones change and shift their channels to such an extent that measurements of discharge or stage made at one time are not comparable with others made at other times. In comparing records of different dates collected along the Mississippi River, the effect of the changes in conditions which affect the flow must be kept in mind, and there will always be an element of uncertainty which requires caution in their use. Although these comparisons may give a general idea of the conditions, they will always be open to question and should be used with great caution.

NOTE.—This paper by C. S. Jarvis, M. Am. Soc. C. E., was published in March, 1942, *Proceedings*. Discussion has appeared in *Proceedings*, as follows: September, 1942, by Messrs. Robert Follansbee, and R. W. Davenport; and November, 1942, by Charles Senour, M. Am. Soc. C. E.

[28] Cons. Hydr. Engr., U. S. Geological Survey, Washington, D. C.

[28a] Received by the Secretary November 4, 1942.

[2] "Report on the Physics and Hydraulics of the Mississippi River," by A. A. Humphreys and Henry L. Abbot, *Professional Paper No. 13*, Corps of Engrs., U. S. Army, 1876, p. 130.